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US Army Corps
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ENGINEERING AND DESIGN

NAVSTAR Global Positioning System Surveying

ENGINEER MANUAL

DEPARTMENT OF THE ARMY EM 1110-1-1003
U.S. Army Corps of Engineers
CECW-EP **Washington, DC 20314-1000**

Manual
No. 1110-1-1003 1 August 1996

Engineering and Design
NAVSTAR GLOBAL POSITIONING SYSTEM SURVEYING

1. Purpose. This manual provides technical specifications and procedural guidance for surveying with the NAVSTAR Global Positioning System (GPS). It is intended for use by engineering, topographic, or construction surveyors performing surveys for civil works and military construction projects. Procedural and quality control standards are defined to establish Corps-wide uniformity in GPS survey performance and GPS Architect-Engineer (A-E) contracts.

2. Applicability. This manual applies to HQUSACE elements, major subordinate commands (MSC), districts, laboratories, and field operating activities (FOA) having responsibility for the planning, engineering and design, operations, maintenance, construction, and related real estate and regulatory functions of civil works and military construction projects. It applies to GPS survey performance by both hired labor forces and contracted survey forces. It is also applicable to surveys performed or procured by local interest groups under various cooperative or cost-sharing agreements.

3. General. The NAVSTAR GPS has significantly modified many traditional survey practices found in all aspects of surveying and mapping work. The NAVSTAR GPS, operating in a differential or relative survey mode, is capable of providing far more accurate positions of either static monuments or moving platforms at costs far less than those for conventional survey methods. The goal of this manual is to ensure that GPS survey procedures are efficiently and uniformly practiced to attain more accurate and cost-effective surveying and mapping execution throughout the Corps of Engineers.

FOR THE COMMANDER:

ROBERT H. GRIFFIN
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-1-1003, dated 31 December 1994.

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Chapter 1
Introduction

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1-3. References

Required and related publications are listed in Appendix A.

1-4. Explanation of Abbreviations and Terms

GPS surveying terms and abbreviations used in this manual are explained in the Glossary (Appendix B).

1-5. Trade Name Exclusions

The citation or illustration in this manual of trade names of commercially available GPS products, including other auxiliary surveying equipment, instrumentation, and adjustment software, does not constitute official endorsement or approval of the use of such products.

1-6. Accompanying Guide Specification

A guide specification for the preparation of A-E contracts for GPS survey services is contained in Appendix G.

1-7. Background

GPS surveying is a process by which highly accurate, three-dimensional (3D) point positions are determined from signals received from NAVSTAR satellites. GPS-derived positions may be used to provide the primary reference control monument locations for engineering and construction projects, from which detailed site plan topographic mapping, boundary demarcation, and construction alignment work may be performed using conventional surveying instruments and procedures. GPS surveying also has application in the continuous positioning of marine floating plants. GPS surveying can also be used for input to Geographic Information System (GIS) and mapping projects.

1-8. Scope of Manual

This manual deals primarily with the use of differential carrier phase GPS survey techniques for establishing and/or extending project construction or boundary control. Both static and kinematic survey methods are covered, along with related GPS data reduction, post-processing, and adjustment methods. Differential code phase GPS positioning and navigation methods supporting hydrographic surveying and dredge control are covered to a lesser extent (see EM 1110-2-1003 for further information on hydrographic surveying with GPS). Kinematic (or dynamic) real-time differential carrier phase GPS surveying applications are covered in detail in this manual. Absolute GPS point positioning methods (i.e., nondifferential) are also described since these techniques have an application in some USACE surveying and mapping projects.

a. This manual is intended to be a comprehensive reference guide for differential carrier phase GPS surveying, whether performed by in-house, hired-labor forces, contracted forces, or combinations thereof. General planning criteria, field and office execution procedures, and required accuracy specifications for performing differential GPS surveys in support of USACE engineering, construction, operations, planning, and real estate activities are provided. Accuracy specifications, procedural criteria, and quality control requirements contained in this manual shall be directly referenced in the scopes of work for A-E survey services or other third-party survey services. This is intended to ensure that uniform and standardized procedures are followed by both hired-labor and contract service sources throughout USACE.

b. The primary emphasis of the manual centers on performing second- and third-order accuracy surveys. This accuracy level will provide adequate reference control from which supplemental real estate, engineering, construction layout surveying, and site plan topographic mapping work may be performed using conventional survey techniques. Therefore, the survey criteria given in this manual will not necessarily meet the Federal Geodetic Control Subcommittee (FGCS) standards and specifications required for the National Geodetic Reference System (NGRS). However, it should be understood that following the methods and procedures given in this manual will give final results generally equal to or exceeding FGCS second-order relative accuracy criteria. This is adequate for the majority of USACE projects.

c. Chapter 12 herein on GPS cost estimating is intended to assist those USACE Commands which primarily contract out survey services. Refer to Appendix G for further information concerning the contracting of GPS services.

d. This manual briefly covers the theory and physical concepts of NAVSTAR GPS positioning. Consult the related publications in Appendix A for further information.

1-9. Life Cycle Project Management Applicability

Project control established by GPS survey methods may be used through the entire life cycle of a project, spanning decades in many cases. During initial reconnaissance surveys of a project, control established by GPS should be permanently monumented and situated in areas that are conducive to the performance or densification of subsequent surveys for contract plans and specifications, construction, and maintenance. During the early planning phases of a project, a comprehensive survey control plan should be developed which considers survey requirements over a project's life cycle, with a goal of eliminating duplicative or redundant surveys to the maximum extent possible.

1-10. Metrics

Metric units are used in this manual. Metric units are commonly used in geodetic surveying applications, including the GPS survey work covered herein. GPS-derived geographical or metric Cartesian coordinates are generally transformed to non-SI units of measurements for use in local project reference and design systems, such as State Plane Coordinate System (SPCS) grids. In all cases, the use of metrics shall follow local engineering and construction practices. Non-SI/metric equivalencies are noted where applicable, including the critical--and often statutory--distinction between the U.S. Survey Foot (1,200/3,937 m exactly) and International Foot (30.48/100 m exactly) conversions.

1-11. Manual Development and Proponency

The HQUSACE proponent for this manual is the Surveying and Analysis Section, General Engineering Branch, Civil Works Directorate. The manual was developed by the U.S. Army Topographic Engineering Center (USATEC) during the period 1992-1994 under the Civil Works Guidance Update Program, U.S. Army Engineer Waterways Experiment Station. Primary technical authorship and/or review was provided by the U.S. Army Engineer Districts, Pittsburgh, Tulsa, Detroit, New Orleans, and St. Louis. Recommended corrections or modifications to this manual should be directed to HQUSACE, ATTN: CECW-EP-S, 20 Massachusetts Ave. NW, Washington, DC 20324-1000.

1-12. Distribution

Copies of this document or any other Civil Works Criteria Documents can be obtained from: U.S. Army Corps of Engineers, Publications Depot, 2803 52nd Ave, Hyattsville, MD 20781-1102, Phone: (301) 394-0081.

1-13. Further Information

Further information on the technical contents of this manual can be obtained from:

U.S. Army Topographic Engineering Center
ATTN: CETEC-TD-AG
Surveying Division
7701 Telegraph Road
Alexandria, VA 22315-3864

Phone: (703) 428-6766
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Chapter 2
Operational Theory of NAVSTAR GPS

This chapter provides a general overview of the basic operating principles and theory of the NAVSTAR GPS. The references listed in Appendix A should be used for more detailed background of all the topics covered in this chapter.

2-1. Global Positioning System (GPS)

The NAVSTAR GPS is a passive, satellite-based, navigation system operated and maintained by the Department of Defense (DoD). Its primary mission is to provide passive global positioning/navigation for land-, air-, and sea-based strategic and tactical forces. A GPS receiver is simply a range measurement device; distances are measured between the receiver antenna and the satellites, and the position is determined from the intersections of the range vectors. These distances are determined by a GPS receiver which precisely measures the time it takes a signal to travel from the satellite to the station. This measurement process is similar to that used in conventional pulsing marine navigation systems and in phase comparison electronic distance measurement (EDM) land surveying equipment.

a. GPS operating and tracking modes. There are basically two general operating modes from which GPS-derived positions can be obtained: absolute positioning and relative or differential positioning. Within each of these two modes, range measurements to the satellites can be performed by tracking either the phase of the satellite's carrier signal or the pseudo-random noise codes modulated on the carrier signal. In addition, GPS positioning can be performed with the receiver operating in a static or dynamic (kinematic) environment. This variety of operational options results in a wide range of accuracy levels which may be obtained from the NAVSTAR GPS. Accuracies can range from 100 m down to the sub-centimeter level, as shown in Figure 2-1. Increased accuracies to the sub-centimeter level require additional observing time and, until recently, could not be achieved in real time. Selection of a particular GPS operating and tracking mode (i.e., absolute, differential, code, carrier, static, kinematic, or combinations thereof) depends on the user application. USACE survey applications typically require differential positioning using carrier phase tracking. Some dredge control and hydrographic applications can use differential code measurements. Absolute modes are rarely used for geodetic surveying applications except when worldwide reference control is being established.

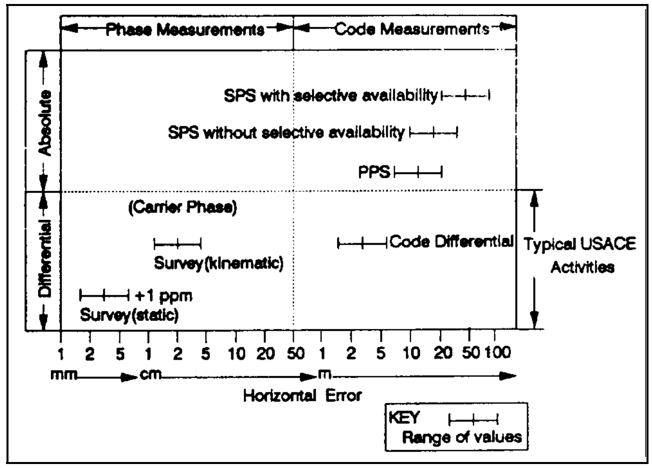

Figure 2-1. GPS operating modes and accuracies

b. Absolute positioning. The most common military and civil (i.e., commercial) application of GPS is "absolute positioning" for real-time navigation. When operating in this passive, real-time navigation mode, ranges to NAVSTAR satellites are observed by a single receiver positioned on a point for which a position is desired. This receiver may be positioned to be stationary over a point (i.e., static, Figure 2-2) or in motion (i.e., kinematic positioning, such as on a vehicle, aircraft, missile, or backpack). Two levels of absolute positioning accuracy may be obtained from the NAVSTAR GPS. These are called the (1) Standard Positioning Service (SPS) and (2) Precise Positioning Service (PPS).

(1) Using the SPS, the user is able to achieve real-time 3D absolute point positioning on the order of 100 m. The SPS is the GPS signal that the DoD authorizes to civil users. This level of accuracy, achievable by the civil user, is due to the deliberate degradation of the GPS signal by the DoD for national security reasons. DoD degradation of the GPS signal is referred to as "Selective Availability" or S/A. DoD has also implemented Anti-Spoofing or A-S which will deny the SPS user the more accurate P-code. S/A and A-S will be discussed further in Chapter 5.

(2) Use of the PPS requires authorization by DoD to have a decryption device capable of deciphering the encrypted GPS signals. USACE is an authorized user; however, actual use of the equipment has security implications. Real-time 3D absolute positional accuracies of 16-20 m are attainable through use of the PPS.

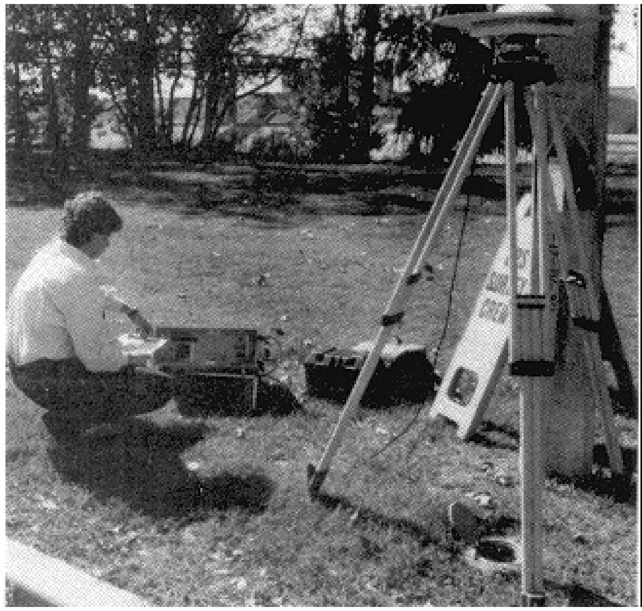

Figure 2-2. Performing static differential GPS surveys

(3) With certain specialized GPS receiving equipment, data processing refinements, and long-term static observations, absolute positional coordinates may be determined to accuracy levels less than a meter. Applications of this are usually limited to worldwide geodetic reference surveys.

(4) These absolute point positioning accuracy levels are not suitable for USACE surveying applications other than rough reconnaissance work or general vessel navigation. They may be useful for some military topographic surveying applications (e.g., artillery surveying).

c. Differential or relative GPS positioning. Differential positioning is simply a process of measuring the differences in coordinates between two receiver points, each of which is simultaneously observing/measuring satellite code ranges and/or carrier phases from the NAVSTAR GPS constellation. The process actually involves the measurement of the difference in ranges between the satellites and two or more ground observing points. The range measurement is performed by a phase difference comparison, using either the carrier phase or code phase. The basic principle is that the absolute positioning errors at the two receiver points will be approximately the same for a given instant. The resultant accuracy of these coordinate differences is at the meter level for code phase observations and at the centimeter level for carrier phase tracking. These coordinate differences are usually expressed as 3D "baseline vectors," which are comparable to conventional survey azimuth/distance measurements. Differential GPS (DGPS) positioning can be performed in either a static or kinematic mode. Further information on DGPS can be found in Chapter 6.

2-2. NAVSTAR Program Background

A direct product of the "space race" of the 1960's, the NAVSTAR GPS is actually the result of the merging of two independent programs that were begun in the early 1960's: the U.S. Navy's TIMATION Program and the U.S. Air Force's 621B Project. Another system similar in basic concept to the current NAVSTAR GPS was the U.S. Navy's TRANSIT program, which was also developed in the 1960's. Currently, the entire system is maintained by the NAVSTAR GPS Joint Program Office (JPO), a North Atlantic Treaty Organization (NATO) multiservice type organization. DoD originally designed the NAVSTAR GPS to provide sea, air, and ground forces of the United States and members of NATO with a unified, high-precision, all-weather, worldwide, real-time positioning system. Mandated by Congress, GPS is freely used by both the military and civilian public for real-time absolute positioning of ships, aircraft, and land vehicles, as well as highly precise differential point positioning.

2-3. NAVSTAR System Configuration

The NAVSTAR GPS consists of three distinct segments: the space segment (satellites), the control segment (ground tracking and monitoring stations), and the user segment (air-, land-, and sea-based receivers). See Figure 2-3 for a representation of the basic GPS system segments.

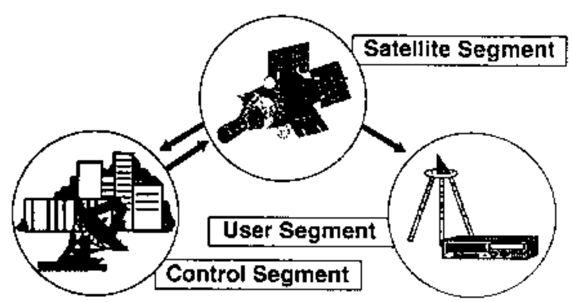

Figure 2-3. GPS system segments

a. Space segment. The space segment consists of all GPS satellites in orbit. The first generation of satellites was the Block I or developmental. Several of these are still operational. A full constellation of Block II or production satellites is presently being put into orbit using Delta II launch vehicles. *The full 24-satellite constellation is scheduled to be in orbit by early FY94.* When this full constellation is implemented, there will be 24 Block II operational satellites (21 primary with 3 active on-orbit spares). There will be four satellites in each of six orbital planes inclined at 55 deg to the equator. The satellites will be at altitudes of 10,898 nm (20,183 km), and have 11-hr-56-minute orbital periods. The three active spares will be transparent to the user on the ground; i.e., the user will not be able to tell which are operational satellites and which are spares. A procurement action for Block IIR (R is for replacement) satellites is underway, thus ensuring full system performance through the year 2025. Figure 2-4 illustrates some of the common design characteristics of the NAVSTAR GPS fully configured Block IIR constellation.

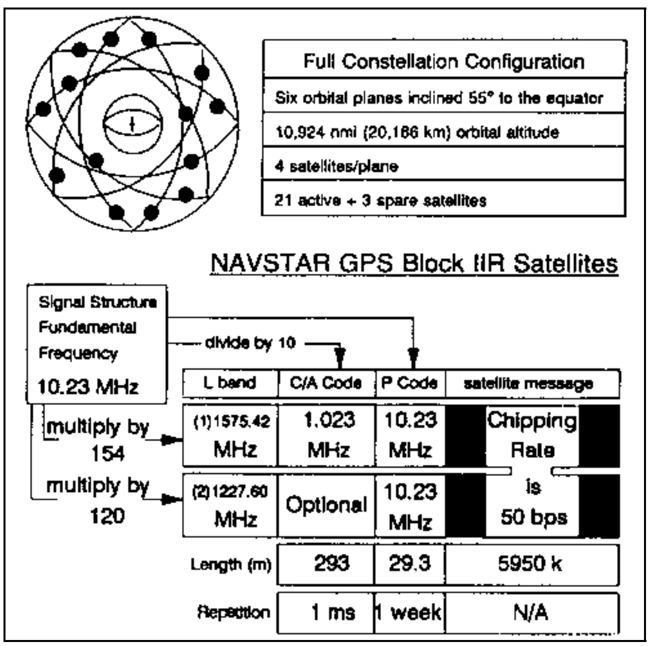

Figure 2-4. NAVSTAR GPS Block IIR constellation

b. Control segment. The GPS control segment consists of five tracking stations located throughout the world (Figure 2-5). These stations are in Hawaii, Colorado, Ascension Island, Diego Garcia, and Kwajalein. The information obtained from tracking the satellites is used in controlling the satellites and predicting their orbits. Three of the stations (Ascension, Diego Garcia, and Kwajalein) are used for transmitting information back to the satellites. The Master Control Station is located at Colorado Springs, Colorado. All data from the tracking stations are transmitted to the Master Control Station where they are processed and analyzed. Ephemerides, clock corrections, and other message data are then transmitted back to the three stations for subsequent transmittal back to the satellites. The Master Control Station is also responsible for the daily management and control of the GPS satellites and the overall control segment.

c. User segment. The user segment represents the ground-based receiver units that process the NAVSTAR satellite signals and arrive at a position of the user. It consists of both military and civil activities for an almost unlimited number of applications in a variety of air-, sea-, or land-based platforms. Land surveying applications (including those of USACE) represent a small percentage of current and potential GPS users.

2-4. GPS Broadcast Frequencies and Codes

Each NAVSTAR satellite transmits signals on two L-band frequencies, designated as L1 and L2. The L1 carrier frequency is 1575.42 megahertz (MHz) and has a wavelength of approximately 19 centimeters (cm). The L2 carrier frequency is 1227.60 MHz and has a wavelength of approximately 24 cm. The L1 signal is modulated with a Precise Code (P-code) and a Coarse Acquisition Code (C/A-code). The L2 signal is modulated with only the P-code. Each satellite carries precise atomic clocks to generate the timing information needed for precise positioning. A navigation message is also transmitted on both frequencies. This message contains ephemerides, clock correction and coefficients, health and status of satellites, almanacs of all GPS satellites, and other general information.

a. Pseudo-random noise. The modulated C/A- and P-codes are referred to as pseudo-random noise (PRN). This pseudo-random code is actually a sequence of very precise time marks that permit the ground receivers to compare and compute the time of transmission between the satellite and ground station. From this transmission time, the range to the satellite can be derived. This is the basis behind GPS range measurements. The C/A-code pulse intervals are approximately every 300 m in range and the more accurate P-code every 30 m.

b. Pseudo-ranges. A pseudo-range is the time delay between the satellite clock and the receiver clock, as determined from C/A- or P-code pulses. This time

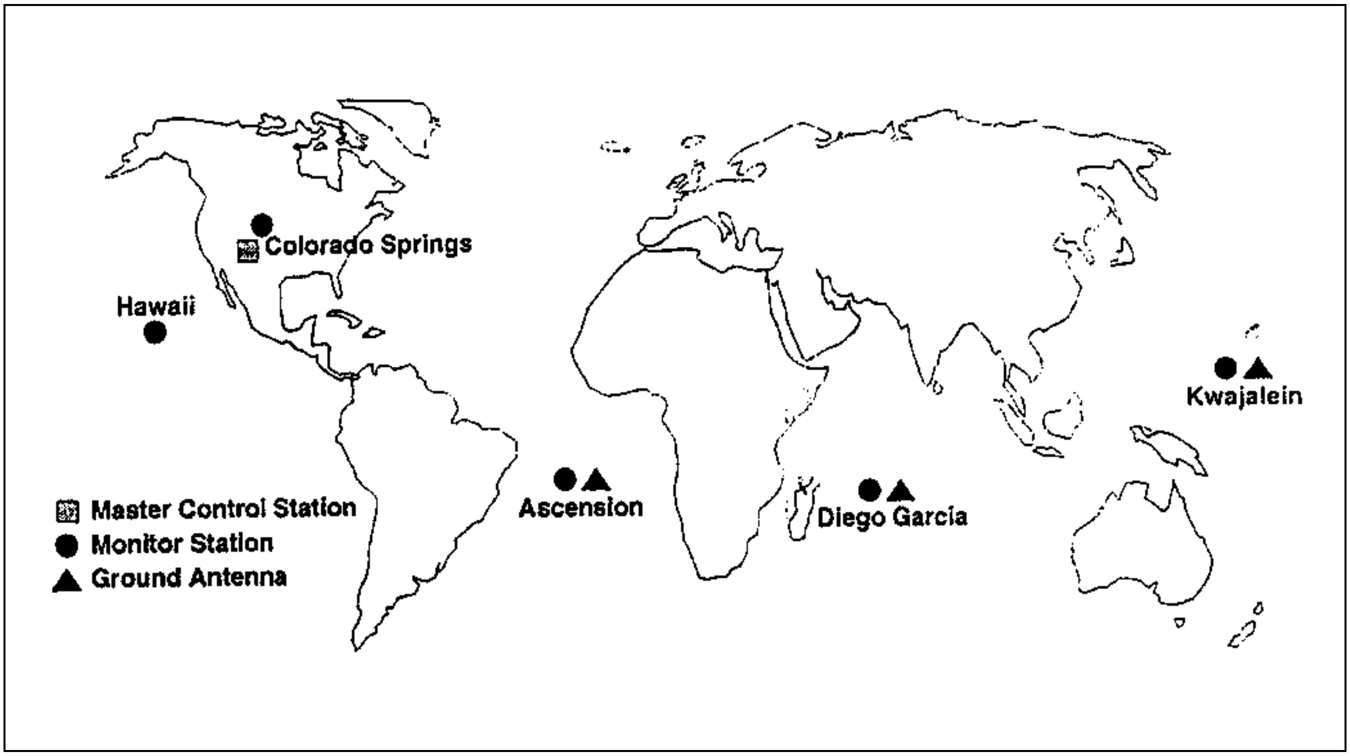

Figure 2-5. GPS control station network

difference equates to the range measurement but is called a pseudo-range since at the time of the measurement, the receiver clock is not synchronized to the satellite clock. In most cases, an absolute 3D real-time navigation position can be obtained by observing at least four simultaneous pseudo-ranges.

c. SPS. The SPS uses the less precise C/A-code pseudo-ranges for real-time GPS navigation. Due to deliberate DoD degradation of the C/A-code accuracy, 100 m in horizontal and 156 m in vertical accuracy levels result. These accuracy levels are adequate for most civil or nonmilitary applications, where only approximate real-time navigation is required.

d. PPS. The PPS is the fundamental military real-time navigation use of GPS. Pseudo-ranges are obtained using the higher pulse rate (i.e., higher accuracy) P-code on both frequencies (L1 and L2). Real-time 3D accuracies at the 16-m level (and 10 m horizontal) can be achieved with the PPS. The P-code is encrypted to prevent unauthorized civil or foreign use. This encryption will require a special key to obtain this 16-m accuracy. These accuracies are adequate for some USACE surveying and mapping projects (i.e. GIS database input).

e. Carrier phase measurements. Carrier frequency tracking measures the phase differences between the Doppler shifted satellite and receiver frequencies. The phase differences are continuously changing due to the changing satellite earth geometry. However, such effects are resolved in the receiver and subsequent data post-processing. When carrier phase measurements are observed and compared between two stations (i.e., relative or differential mode), baseline vector accuracy between the stations below the centimeter level is attainable in three dimensions. New receiver technology and processing techniques have allowed for carrier phase measurements to be used in real-time centimeter positioning.

2-5. GPS Broadcast Messages and Ephemeris Data

Each NAVSTAR GPS satellite periodically broadcasts data concerning clock corrections, system/satellite status, and most critically, its position or ephemeris data. There are two basic types of ephemeris data: broadcast and precise.

a. Broadcast ephemerides. The broadcast ephemerides are actually predicted satellite positions broadcast within the navigation message that are transmitted from the satellites in real time. The ephemerides can be acquired in real time by a receiver capable of acquiring either the C/A- or P-code. The broadcast ephemerides are computed using past tracking data of the satellites. The satellites are tracked continuously by the monitor stations to obtain more recent data to be used for the orbit predictions. The data are analyzed by the Master Control Station, and new parameters for the satellite orbits are transmitted back to the satellites. This upload is performed daily with new predicted orbital elements transmitted every hour by the navigation message.

b. *Precise ephemerides.* The precise ephemerides are based on actual tracking data that are post-processed to obtain the more accurate satellite positions. These ephemerides are available at a later date and are more accurate than the broadcast ephemerides because they are based on actual tracking data and not predicted data. Nonmilitary users can obtain this information from the National Geodetic Survey (NGS) or from private sources that maintain their own tracking networks and provide information for a fee. For most USACE survey applications, the broadcast ephemerides are adequate to obtain the needed accuracies.

c. See Appendix D for sources of GPS information and its status.

Chapter 3
GPS Applications in USACE

3-1. General

Currently, surveyors use GPS to increase their efficiency, productivity, and to produce more accurate results. GPS can be used for real estate surveys, regulatory enforcement actions, horizontal and vertical control densification, structural deformation studies, airborne photogrammetry, dynamic positioning and navigation for hydrographic survey vessels and dredges, hydraulic study/survey location, river/floodplain cross-section location, core drilling location, environmental studies, levee overbank surveys, and levee profiling. Future construction uses of dynamic GPS are unlimited: levee grading and revetment placement, disposal area construction, grade control, etc. Additionally, GPS has application in developing various levels of GIS spatial data. A few of these applications are briefly described in this chapter.

3-2. Project Control Densification

Establishing or densifying project control with GPS is often cost-effective, faster, more accurate, and more reliable than conventional survey methods. The quality control statistics and large number of redundant measurements in GPS networks help to ensure reliable results. Field operations to perform a GPS survey are relatively easy and can generally be performed by one person per receiver. GPS is particularly attractive for control networks as compared with conventional surveys because intervisibility is not required between adjacent stations.

3-3. Geodetic Control Densification

GPS can be used for wide-area high-order geodetic control densification. GPS provides very precise point positioning (when used in a relative mode), producing baseline results on the order of 5 to 10 ppm under average conditions.

3-4. Vertical Control Densification

GPS uses the World Geodetic System of 1984 (WGS 84) ellipsoid as the optimal mathematical model describing the shape of the earth on an ellipsoid of rotation. There is no direct mathematical relation between heights obtained from GPS and orthometric elevations obtained from conventional spirit leveling. However, a model can be determined from benchmark data and corresponding GPS data. This model can then be used to derive the unknown orthometric heights of stations occupied during a GPS observation period to densify supplemental small-scale topographic mapping. Geoid modeling software also exists and is used to determine orthometric heights from GPS. Extreme caution should be taken in using GPS for vertical densification. The procedures for vertical densification are described in further detail in Chapter 6.

3-5. Structural Deformation Studies

GPS survey techniques can be used to monitor the motion of points on a structure relative to stable monuments. This can be done with an array of antennae positioned at selected points on the structure and on remote stable monuments. Baselines are formulated between the occupied points to monitor differential movement. The relative precision of the measurements is on the order of ±5 mm over distances averaging between 5 and 10 km. Measurements can be made on a continuous basis. A GPS structural deformation system can operate unattended and is relatively easily installed and maintained.

3-6. Photogrammetry

The use of an airborne GPS receiver employing on-the-fly (OTF) techniques combined with specialized photogrammetric procedures has the potential to significantly reduce the amount of ground control for typical photogrammetric projects. Currently, these projects require a significant amount of manpower and monetary resources for the establishment of the control points. Therefore, the use of this GPS Controlled Photogrammetry (GCP) technology in the USACE civil works programs should reduce the production costs associated with large scale maps. The benefits of GCP will be realized in the savings estimation based on the premise that most of the USACE photogrammetry activities require USACE personnel to do much planning and surveying in preparation for the actual photogrammetry flight, and the GCP procedure has the potential for the reduction, or even elimination, of this surveying activity. Tests have shown that ground control coordinates can be developed from an airborne platform using adapted GPS kinematic techniques to centimeter-level precision in all three axes if system-related errors are minimized and care is taken in conduct of the GPS and photogrammetric portions of the procedures. High quality photogrammetric results can also be achieved with DGPS based on carrier-smoothed code phase positioning.

3-7. Dynamic Positioning and Navigation

Dynamic, real-time GPS code and carrier phase positioning of construction and surveying platforms has the potential for revolutionizing many current USACE design and construction functions. This includes dredge control systems, site investigation studies/surveys, horizontal and vertical construction placement, hydraulic studies, or any other activity requiring dimensional control. Real-time, centimeter-level 3D (based on the WGS 84 Ellipsoid) control may be achieved using carrier phase differential GPS; this method can be used for any type of construction or survey platform (e.g., dredges, graders, survey vessels, etc.). This method is discussed further in Chapter 6.

3-8. GIS Integration

A GIS is an effective means to correlate and store diverse information on natural or man-made characteristics of geographic positions. In order for a GIS to be reliably oriented, it should be based on a coordinate system. A standardized GIS network enables a more accurate exchange of GIS information between databases. In recent years, GPS has demonstrated its efficiency, cost effectiveness, and accuracy in precise surveying and mapping support.

Chapter 4
GPS Reference Systems

4-1. General

In order to fully understand GPS, and its positional information, it is important to understand the reference system on which it is based. The GPS satellites are referenced to the WGS 84 ellipsoid. For surveying purposes, this earth-centered WGS 84 coordinate system must be converted (i.e., transformed) to a user-defined ellipsoid/datum, such as the Clarke 1866 (North American Datum of 1927 (NAD 27)) or Geodetic Reference System of 1980 (GRS 80) reference ellipsoids. Differential positioning provides this conversion by locating one of the receivers at a known point on the user's datum. This chapter deals with GPS reference systems and datums to which GPS coordinates can be transformed.

4-2. Geodetic Coordinate Systems

The absolute positions obtained directly from GPS pseudo-range measurements are based on the 3D, earth-centered WGS 84 ellipsoid. Coordinate outputs are on a Cartesian system (X, Y, and Z) relative to an Earth Centered Earth Fixed (ECEF) Rectangular Coordinate System having the same origin as the WGS 84 ellipsoid, i.e. geocentric. This geocentric X-Y-Z coordinate system should not be confused with the X-Y plane coordinates established on local grids; local systems usually have entirely different definitions, origins, and orientations which require certain transformations to be performed. WGS 84 Cartesian coordinates can be easily converted into WGS 84 ellipsoid coordinates (i.e., ϕ, λ, and h, geodetic latitude, longitude, and height, respectively).

4-3. WGS 84 Reference Ellipsoid

a. The origin of the WGS 84 Cartesian system is the earth's center of mass. The Z-axis is parallel to the direction of the Conventional Terrestrial Pole (CTP) for polar motion, as defined by the Bureau International Heure (BIH), and equal to the rotation axis of the WGS 84 ellipsoid. The X-axis is the intersection of the WGS 84 reference meridian plane and the CTP's equator, the reference meridian being parallel to the zero meridian defined by the BIH and equal to the X-axis of the WGS 84 ellipsoid. The Y-axis completes a right-handed, earth-centered, earth-fixed orthogonal coordinate system, measured in the plane of the CTP equator 90 deg east of the X-axis and equal to the Y-axis of the WGS 84 ellipsoid. This system is illustrated in Figure 4-1.

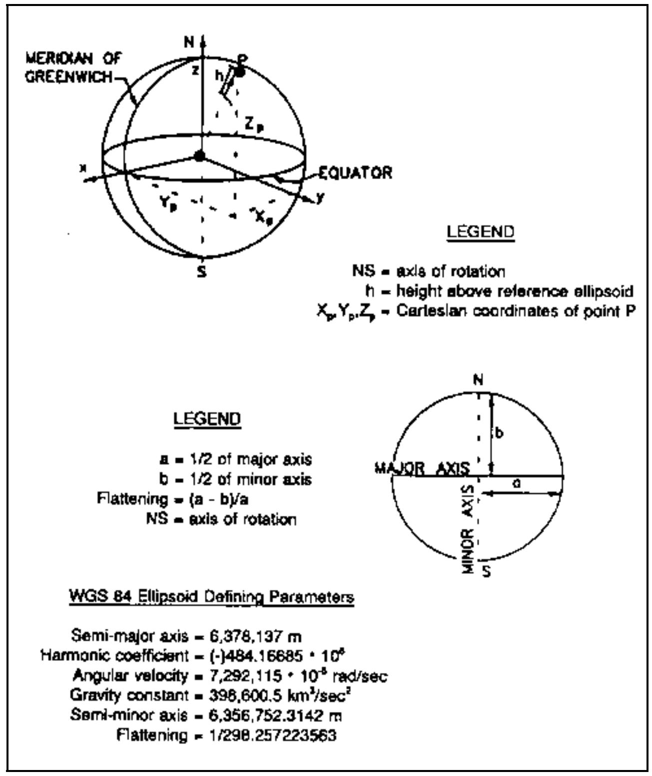

Figure 4-1. GPS WGS 84 reference ellipsoid

b. Prior to development of WGS 84, there were several reference ellipsoids and interrelated coordinate systems (datums) that were used by the surveying community. Table 4-1 lists just a few of these systems, some of which are widely used even today.

Table 4-1
Reference Ellipsoids and Related Coordinate Systems

Reference Ellipsoid	Coordinate System (Datum)
Clarke 1866	NAD 27
WGS 72	WGS 72
GRS 80	NAD 83
WGS 84	WGS 84

4-4. Horizontal Positioning Datums

One USACE application of differential GPS surveying is in densifying military construction and civil works project control. This densification is usually done relative to an existing datum (NAD 27, NAD 83, or local). Even though GPS measurements are made relative to the WGS 84 ellipsoidal coordinate system, coordinate differences (i.e., baseline vectors) on this system can, for practical engineering purposes, be used directly on any local user datum. Thus, a GPS-coordinated WGS 84 baseline can be directly used on an NAD 27, NAD 83, or even a local project datum. Minor variations between these datums will be minimal when GPS data are adjusted to fit between local datum stations. Such assumptions may not be valid when high-order NGRS network densification work is being performed.

a. North American Datum of 1927 (NAD 27). NAD 27 is a horizontal datum based on a comprehensive adjustment of a national network of traverse and triangulation stations. NAD 27 is a best fit for the continental United States. The fixed datum reference point is located at Meades Ranch, Kansas. The longitude origin of NAD 27 is the Greenwich Meridian with a south azimuth orientation. The original network adjustment used 25,000 stations. The relative precision between initial point monuments of NAD 27 is by definition 1:100,000, but coordinates on any given monument in the network contain errors of varying degrees. As a result, relative accuracy between points on NAD 27 may be far less than the nominal 1:100,000. The reference units for NAD 27 are U.S. Survey Feet.

b. North American Datum of 1983 (NAD 83). NAD 83 uses many more stations and observations than NAD 27, including some satellite-derived coordinates, to readjust the national network (a total of approximately 250,000 stations were used). The longitude origin of NAD 83 is the Greenwich Meridian with a north azimuth orientation. NAD 83 has an average precision of 1:300,000. NAD 83 is based upon the GRS 80, an earth-centered reference ellipsoid, and for most practical purposes is equivalent to WGS 84, which is currently the best available geodetic model of the shape of the earth surface worldwide. The reference units for NAD 83 are meters.

c. HARNs Network Survey Datum. The nationwide horizontal reference network was redefined in 1983 and readjusted in 1986 by the NGS. It is known as the North American Datum of 1983, adjustment of 1986, and is referred to as NAD 83 (86). It is accurate to 1 part in 100,000 which normally satisfies USACE surveying, mapping, and related spatial database requirements. USACE adopted this datum on 5 March 1990. Since that time, several states and the NGS have begun developing High Accuracy Reference Networks (HARNs) for surveying, mapping, and related spatial database projects. These networks, developed exclusively with GPS, are accurate to 1 part in 1,000,000. HARNs have a slightly different coordinate, usually within one meter of those in NAD 83 (86), resulting in two coordinate values for the same survey point. Since the confusion and potential litigation inherent with multiple coordinates with the same point can adversely impact design, construction, boundary location, and other functions, use of HARNS is not recommended.

d. Geodetic survey datums. GPS uses the WGS 84 reference ellipsoid for geodetic survey purposes. GPS routinely provides differential horizontal positional results on the order of 1 ppm, compared to the accepted results of 1:300,000 for NAD 83 and (approximately) 1:100,000 for NAD 27. Even though GPS has such a high degree of precision, it provides only coordinate differences; therefore, ties to the national network to obtain coordinates of all GPS stations must be done without distorting the established control network (i.e., degrade the GPS-derived vectors during the adjustment). Generally, on midsize survey projects, three or more horizontal control stations from the national network can be used during the GPS observation scheme. In order to facilitate a tie between GPS and existing networks for horizontal control, an adjustment of the whole network scheme (all control and GPS-derived points) should be completed. There are many commercial software packages that can be used to perform this adjustment. Once a network adjustment meets the accuracy requirement, those values should not be readjusted with additional points or observations.

e. Local project datums. Several projects can be based on local project datums. These local datums might be accurate within a small area, but can become distorted over larger areas. Most local project datums are not connected to any other datums, but can be tied to outside control and related and transformed to another datum. It is important to understand how this local datum was established in order to relate it or perform a transformation to some other datum.

f. State Plane Coordinate System. The SPCS was developed by the NGS to provide a planar representation of the earth's surface. To properly relate spherical coordinates (ϕ,λ) to a planar system (Northings and Eastings), a developable surface must be constructed. A developable surface is defined as a surface that can be expanded without stretching or tearing. The two most common developable surfaces or map projections used in surveying and mapping are the cone and cylinder. The projection of choice is dependent on the north-south or east-west extent of the region. Areas with limited east-west dimensions and elongated north-south extent utilize the Transverse Mercator projection. Areas with limited north-south dimensions and elongated east-west extent utilize the

Lambert projection. For further information on the State Plane Coordinate System see EM 1110-1-1004.

4-5. Orthometric Elevations

Orthometric elevations are those corresponding to the earth's irregular geoidal surface. Measured differences in elevation from spirit leveling are generally relative to geoidal heights--a spirit level bubble (or pendulum) positions the instrument normal to the direction of gravity, and thus parallel with the local slope of the geoid. Elevation differences between two points are orthometric differences, a distinction particularly important in river/channel hydraulics. Orthometric heights for the continental United States (CONUS) are generally referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29) or the North American Vertical Datum of 1988 (NAVD 88); however, other vertical datums may be used in some projects (e.g., the International Great Lakes Datum of 1955 (IGLD 55) or International Great Lakes Datum of 1985 (IGLD 85)), which is a dynamic/hydraulic-based datum, not an orthometric datum).

4-6. GPS WGS 84 Ellipsoidal Heights

GPS-determined heights or height differences are referenced to an idealized mathematical ellipsoid, i.e., WGS 84. This WGS 84 ellipsoid differs significantly from the geoid; thus, GPS heights are not the same as the orthometric heights which are needed for standard USACE projects (i.e. local engineering, construction, and hydraulic measurement functions). (See Figure 4-2.) Accordingly, any WGS-84-referenced height obtained using GPS must be transformed to the local orthometric vertical datum. This requires adjusting and interpolating GPS-derived heights relative to fixed orthometric elevations. Such a process may or may not be of suitable accuracy (i.e. reliability) for some engineering and construction work. See Table 6-1 in Chapter 6.

4-7. Orthometric-WGS 84 Elevation Relationship

The relationship between a WGS 84 ellipsoidal height and an orthometric height relative to the geoid can be obtained from the following equation:

$$h = H + N \tag{4-1}$$

where

h = ellipsoidal height

H = elevation (orthometric)

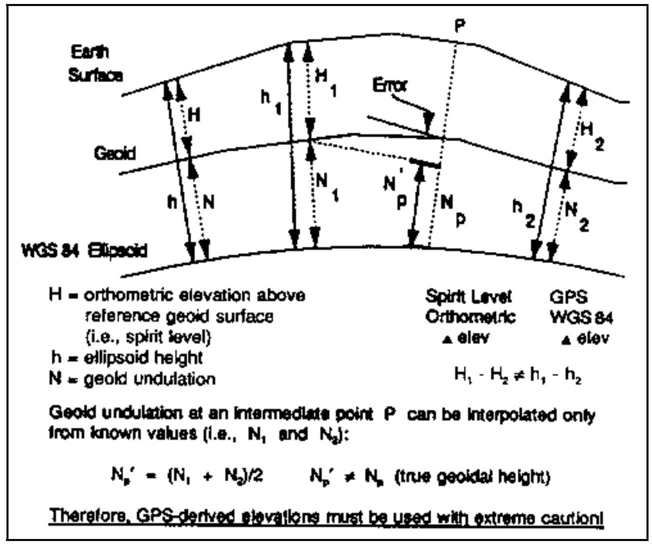

Figure 4-2. GPS ellipsoid heights

N = geoid undulation

a. Due to significant variations in the geoid, even over small distances, elevation differences obtained by GPS cannot be directly equated to orthometric (or spirit level) differences. Geoid modeling techniques are often used to obtain the parameter N in Equation 4-1; however, accuracies may not be adequate for engineering purposes. Some small project areas where the geoid stays fairly constant or local geoid modeling can be performed, elevation differences obtained by GPS can be used. See Chapter 6 for further information on the concept of vertical densification with GPS.

b. GPS surveys can be designed to provide elevations of points on the local vertical datum. This requires connecting to a sufficient number of existing orthometric benchmarks from which the elevations of unknown points can be "best fit" by some adjustment method--usually a least squares minimization. This is essentially an interpolation process and assumes linearity in the geoid slope between two established benchmarks. If the geoid variation is not linear, then the adjusted (interpolated) elevation of an intermediate point will be in error. Depending on the station spacing, location, local geoid undulations, and numerous other factors, the resultant interpolated/adjusted elevation accuracy is usually not suitable for construction surveying purposes; however, GPS-derived elevations may be adequate for small-scale topographic mapping control.

Chapter 5
GPS Absolute Positioning Determination Concepts, Errors, and Accuracies

5-1. General

NAVSTAR GPS determination of a point position on the earth actually uses techniques common to conventional surveying trilateration: an electronic distance measurement resection. The user's receiver simply measures the distance (i.e., ranges) between the earth and the NAVSTAR GPS satellite(s). The user's position is determined by the resected intersection of the observed ranges to the satellites. Each satellite range creates a sphere which forms a circle (approximately) upon intersection with the earth's surface. Given observed ranges to two different satellites, two intersecting circles result from which a horizontal (2D) position on the earth can be computed. Adding a third satellite range creates three spheres, the intersection point of which will provide the X-Y-Z geocentric coordinates of a point. Adding more satellite ranges will provide redundancy in the positioning, which allows adjustment. In actual practice, at least four satellite observations are required in order to resolve timing variations for a 3D position.

5-2. Absolute Positioning

Absolute positioning involves the use of only a single passive receiver at one station location to collect data from multiple satellites in order to determine the station's location. It is not sufficiently accurate for precise surveying or hydrographic positioning uses. It is, however, the most widely used military and commercial GPS positioning method for real-time navigation and location (see paragraph 2-1*b*).

a. The accuracies obtained by GPS absolute positioning are dependent on the user's authorization. The SPS user can obtain real-time point positional accuracies of 100 m. The lower level of accuracies achievable using SPS is due to intentional degradation of the GPS signal by the DoD (S/A). The PPS user (usually a DoD-approved user) can use a decryption device to achieve a point positional (3D) accuracy in the range of 10-16 m with a single-frequency receiver. Accuracies to less than a meter can be obtained from absolute GPS measurements when special equipment and post-processing techniques are employed.

b. Absolute point positioning with the carrier phase. By using broadcast ephemerides, the user is able to use pseudo-range values in real time to determine absolute point positions with an accuracy of between 3 m in the best of conditions and 80 m in the worst. By using a post-processed ephemerides (i.e., precise), the user can expect absolute point positions with an accuracy of near 1 m in the best of conditions and 40 m in the worst.

5-3. Pseudo-Ranging

When a GPS user performs a GPS navigation solution, only an approximate range, or pseudo-range, to selected satellites is measured. In order for the GPS user to determine his/her precise location, the known range to the satellite and the position of those satellites must be known. By pseudo-ranging, the GPS user measures an approximate distance between the antenna and the satellite by correlation of a satellite-transmitted code and a reference code created by the receiver, without any corrections for errors in synchronization between the clock of the transmitter and that of the receiver. The distance the signal has traveled is equal to the velocity of the transmission of the satellite multiplied by the elapsed time of transmission, with satellite signal velocity changes due to tropospheric and ionospheric conditions being considered. Refer to Figure 5-1 for an illustration of the pseudo-ranging concept. (See also paragraph 2-4*a*,*b*.)

a. The accuracy of the positioned point is a function of the range measurement accuracy and the geometry of the satellites, as reduced to spherical intersections with the earth's surface. A description of the geometrical magnification of uncertainty in a GPS-determined point position is Dilution of Precision (DOP), which is discussed in section 5-6*d*(2). Repeated and redundant range observations will generally improve range accuracy. However, the dilution of precision remains the same. In a static mode (meaning the GPS antenna stays stationary), range measurements to each satellite may be continuously remeasured over varying orbital locations of the satellite(s). The varying satellite orbits cause varying positional intersection geometry. In addition, simultaneous range observations to numerous satellites can be adjusted using weighting techniques based on the elevation and pseudo-range measurement reliability.

b. Four pseudo-range observations are needed to resolve a GPS 3D position. (Only three pseudo-range observations are needed for a 2D location.) In practice there are often more than four. This is due to the need to

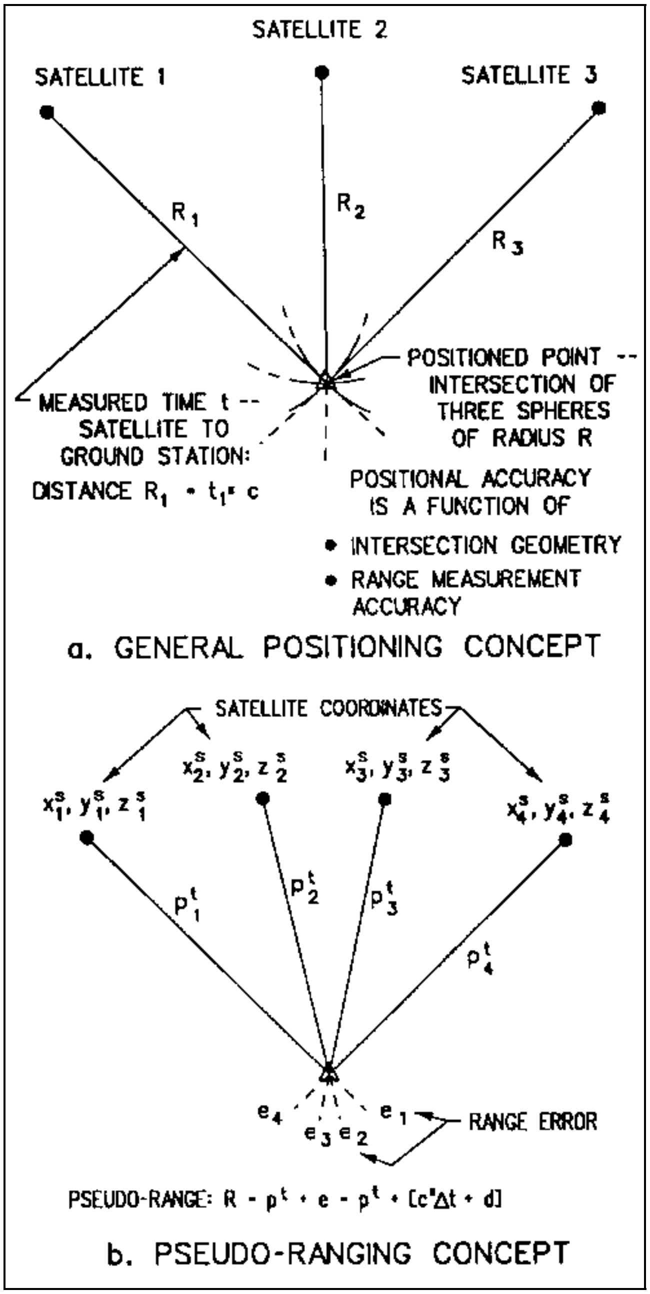

Figure 5-1. GPS satellite range measurement

resolve the clock biases Δt contained in both the satellite and ground-based receiver. Thus, in solving for the X-Y-Z coordinates of a point, a fourth unknown (i.e., clock bias) must also be included in the solution. The solution of the 3D position of a point is simply the solution of four pseudo-range observation equations containing four unknowns, i.e., X, Y, Z, and Δt.

c. A pseudo-range observation is equal to the true range from the satellite to the user ρ^t plus delays due to satellite/receiver clock biases and other effects, as was shown in Figure 5-1.

$$R = p^t + c(\Delta t) + d \tag{5-1}$$

where

R = observed pseudo-range

ρ^t = true range to satellite (unknown)

c = velocity of propagation

Δt = clock biases (receiver and satellite)

d = propagation delays due to atmospheric conditions

These are usually estimated from models.

The true range ρ^t is equal to the 3D coordinate difference between the satellite and user.

$$\rho^t = \left[(X^s - X^u)^2 + (Y^s - Y^u)^2 + (Z^s - Z^u)^2\right]^{\frac{1}{2}} \tag{5-2}$$

where

X^s, Y^s, Z^s = known satellite coordinates from ephemeris data

X^u, Y^u, Z^u = unknown coordinates of user which are to be determined.

When four pseudo-ranges are observed, four equations are formed from Equations 5-1 and 5-2.

$$(R_1 - c\ \Delta t - d_1)^2 = \left(X_1^s - X^u\right)^2 + \left(Y_1^s - Y^u\right)^2 + \left(Z_1^s - Z^u\right)^2 \tag{5-3}$$

$$(R_2 - c\ \Delta t - d_2)^2 = \left(X_2^s - X^u\right)^2 + \left(Y_2^s - Y^u\right)^2 + \left(Z_2^s - Z^u\right)^2 \tag{5-4}$$

$$(R_3 - c\ \Delta t - d_3)^2 = \left(X_3^s - X^u\right)^2 + \left(Y_3^s - Y^u\right)^2 + \left(Z_3^s - Z^u\right)^2 \tag{5-5}$$

$$(R_4 - c\ \Delta t - d_4)^2 = \left(X_4^s - X^u\right)^2 + \left(Y_4^s - Y^u\right)^2 + \left(Z_4^s - Z^u\right)^2 \tag{5-6}$$

In these equations, the only unknowns are X^u, Y^u, Z^u, and Δt. Solving these equations at each GPS update yields the user's 3D position coordinates. Adding more pseudo-range observations provides redundancy to the solution. For instance, if seven satellites are simultaneously observed, seven equations are derived, and still only four unknowns result.

d. This solution is highly dependent on the accuracy of the known coordinates of each satellite (i.e., X^s, Y^s, and Z^s), the accuracy with which the atmospheric delays *d* can be estimated through modeling, and the accuracy of the resolution of the actual time measurement process performed in a GPS receiver (clock synchronization, signal processing, signal noise, etc.). As with any measurement process, repeated and long-term observations from a single point will enhance the overall positional reliability.

5-4. GPS Error Sources

There are numerous sources of measurement error that influence GPS performance. The sum of all systematic errors or biases contributing to the measurement error is referred to as range bias. The observed GPS range, without removal of biases, is referred to as a biased range or "pseudo-range." Principal contributors to the final range error that also contribute to overall GPS error are ephemeries error, satellite clock and electronics inaccuracies, tropospheric and ionospheric refraction, atmospheric absorption, receiver noise, and multipath effects. Other errors include those induced by DoD (Selective Availability (S/A) and Anti-Spoofing (A/S)). In addition to these major errors, GPS also contains random observation errors, such as unexplainable and unpredictable time variation. These errors are impossible to model and correct. The following paragraphs discuss errors associated with absolute GPS positioning modes. Many of these errors are either eliminated or significantly minimized when GPS is used in a differential mode. This is due to the same errors being common to both receivers during simultaneous observing sessions. For a more detailed analysis of these errors, consult one of the technical references listed in Appendix A.

a. *Ephemeris errors and orbit perturbations.* Satellite ephemeris errors are errors in the prediction of a satellite position which may then be transmitted to the user in the satellite data message. Ephemeris errors are satellite dependent and very difficult to completely correct and compensate for because the many forces acting on the predicted orbit of a satellite are difficult to measure directly. Because direct measurement of all forces acting on a satellite orbit is difficult, it is nearly impossible to accurately account or compensate for those error sources when modeling the orbit of a satellite. The previous accuracy levels stated are subject to performance of equipment and conditions. Ephemeris errors produce equal error shifts in calculated absolute point positions.

b. *Clock stability.* GPS relies very heavily on accurate time measurements. GPS satellites carry rubidium and cesium time standards that are usually accurate to 1 part in 10^{12} and 1 part in 10^{13}, respectively, while most receiver clocks are actuated by a quartz standard accurate to 1 part in 10^8. A time offset is the difference between the time as recorded by the satellite clock and that recorded by the receiver. Range error observed by the user as the result of time offsets between the satellite and receiver clock is a linear relationship and can be approximated by the following equation:

$$R_E = T_O * c \tag{5-7}$$

where

R_E = user equivalent range error

T_O = time offset

c = speed of light

(1) The following example shows the calculation of the user equivalent range error (UERE or UR).

T_O = 1 microsecond (μs) = 10^{-06} seconds (s)

c = 299,792,458 m/s

From Equation 5-7:

R_E = (10^{-06} seconds) * 299,792,458 m/s

= 299.79 m = 300 m user equivalent range error

(2) In general, unpredictable transient situations that produce high-order departures in clock time can be

ignored over short periods of time. Even though this may be the case, predictable time drift of the satellite clocks is closely monitored by the ground control stations. Through closely monitoring the time drift, the ground control stations are able to determine second-order polynomials which accurately model the time drift. The second-order polynomial determined by the ground control station to model the time drift is included in the broadcast message in an effort to keep this drift to within 1 millisecond (ms). The time synchronization between the GPS satellite clocks is kept to within 20 nsec (ns) through the broadcast clock corrections as determined by the ground control stations and the synchronization of GPS standard time to the Universal Time Coordinated (UTC) to within 100 ns. Random time drifts are unpredictable, thereby making them impossible to model.

(3) GPS receiver clock errors can be modeled in a manner similar to GPS satellite clock errors. In addition to modeling the satellite clock errors and in an effort to remove them, an additional satellite should be observed during operation to simply solve for an extra clock offset parameter along with the required coordinate parameters. This procedure is based on the assumption that the clock bias is independent at each measurement epoch. Rigorous estimation of the clock terms is more important for point positioning than for differential positioning. Many of the clock terms cancel when the position equations are formed from the observations during a differential survey session.

c. Ionospheric delays. GPS signals are electromagnetic signals and as such are nonlinearly dispersed and refracted when transmitted through a highly charged environment like the ionosphere. Dispersion and refraction of the GPS signal is referred to as an ionospheric range effect because dispersion and refraction of the signal result in an error in the GPS range value. Ionospheric range effects are frequency dependent.

(1) The error effect of ionosphere refraction on the GPS range values is dependent on sunspot activity, time of day, and satellite geometry. GPS operations conducted during periods of high sunspot activity or with satellites near the horizon produce range results with the most error. GPS operations conducted during periods of low sunspot activity, during the night, or with a satellite near the zenith produce range results with the least amount of ionospheric error.

(2) Resolution of ionospheric refraction can be accomplished by use of a dual-frequency receiver (a receiver that can simultaneously record both L1 and L2 frequency measurements). During a period of uninterrupted observation of the L1 and L2 signals, these signals can be continuously counted and differenced. The resultant difference reflects the variable effects of the ionosphere delay on the GPS signal. Single-frequency receivers used in an absolute and differential positioning mode typically rely on ionospheric models that model the effects of the ionosphere. Recent efforts have shown that significant ionospheric delay removal can be achieved using signal frequency receivers.

d. Tropospheric delays. GPS signals in the L-band level are not dispersed by the troposphere, but they are refracted. The tropospheric conditions causing refraction of the GPS signal can be modeled by measuring the dry and wet components. The dry component is best approximated by the following equation:

$$D_C = (2.27 * 0.001) * P_O \tag{5-8}$$

where

D_C = dry term range contribution in zenith direction in meters

P_O = surface pressure in millibar

(1) The following example shows the calculation of average atmospheric pressure P_O = 765 mb:

From Equation 5-8:

D_C = (2.27 * 0.001) * 765 mb

= 1.73655 m = 1.7 m, the dry term range error contribution in the zenith direction

(2) The wet component is considerably more difficult to approximate because its approximation is dependent not just on surface conditions, but also on the atmospheric conditions (water vapor content, temperature, altitude, and angle of the signal path above the horizon) along the entire GPS signal path. As this is the case, there has not been a well-correlated model that approximates the wet component.

e. Multipath. Multipath describes an error affecting positioning that occurs when the signal arrives at the receiver from more than one path. Multipath normally occurs near large reflective surfaces, such as a metal building or structure. GPS signals received as a result of multipath give inaccurate GPS positions when processed. With the newer receiver and antenna designs and sound prior mission planning to eliminate possible causes of multipath, the effects of multipath as an error source can be minimized. Averaging of GPS signals over a period of time can also reduce the effects of multipath.

f. Receiver noise. Receiver noise includes a variety of errors associated with the ability of the GPS receiver to measure a finite time difference. These include signal processing, clock/signal synchronization and correlation methods, receiver resolution, signal noise, and others.

g. Selective Availability (S/A) and Anti-Spoofing (A/S). S/A purposely degrades the satellite signal to create position errors. This is done by dithering the satellite clock and offsetting the satellite orbits. The effects of S/A can be eliminated by using differential techniques discussed further in Chapter 6. A-S is implemented by interchanging the P-code with a classified Y-code. This denies users who do not possess an authorized decryption device. Manufacturers of civil GPS equipment have developed methods such as squaring or cross correlation in order to make use of the P-code when it is encrypted.

5-5. User Equivalent Range Error

The previous sources of errors or biases are principal contributors to overall GPS range error. This total error budget is often summarized as the UERE. As mentioned previously, they can be removed or at least effectively suppressed by developing models of their functional relationships in terms of various parameters that can be used as a corrective supplement for the basic GPS information. Differential techniques also eliminate many of these errors. Table 5-1 lists the more significant sources for errors and biases and correlates them to the segment source.

5-6. Absolute GPS Accuracies

The absolute range accuracies obtainable from GPS are largely dependent on which code (C/A or P) is used to determine positions. These range accuracies (i.e., UERE), when coupled with the geometrical relationships of the satellites during the position determination (i.e., DOP), result in a 3D confidence ellipsoid which depicts uncertainties in all three coordinates. Given the changing satellite geometry and other factors, GPS accuracy is time/location dependent. Error propagation techniques are used to define nominal accuracy statistics for a GPS user.

a. Root mean square error measures. Two-dimensional (2D) (horizontal) GPS positional accuracies are normally estimated using a root mean square (RMS) radial error statistic. A 1-σ RMS error equates to the radius of a circle in which the position has a 63 percent probability of falling. A circle of twice this radius (i.e., 2-σ RMS or 2DRMS) represents (approximately) a 97 percent positional probability circle. This 97 percent probability circle, or 2DRMS, is the most common positional accuracy statistic used in GPS surveying. In some instances, a 3DRMS or 99+ percent probability is used. This RMS error statistic is also related to the positional variance-covariance matrix. (Note that an RMS error statistic represents the radius of a circle and therefore is not preceded by a ± sign.)

Table 5-1
GPS Range Measurement Accuracy

		Absolute Positioning		
Segment Source	Error Source	C/A-code Pseudo-range, m	P-code Pseudo-range, m	Differential Positioning, m (P-code)
Space	Clock stability	3.0	3.0	Negligible
	Orbit perturbations	1.0	1.0	Negligible
	Other	0.5	0.5	Negligible
Control	Ephemeris predictions	4.2	4.2	Negligible
	Other	0.9	0.9	Negligible
User	Ionosphere	3.5	2.3	Negligible
	Troposphere	2.0	2.0	Negligible
	Receiver noise	1.5	1.5	1.5
	Multipath	1.2	1.2	1.2
	Other	0.5	0.5	0.5
1-σ UERE		±12.1	±6.5	±2.0

[a] Without S/A.

b. Probable error measures. 3D GPS accuracy measurements are most commonly expressed by Spherical Error Probable, or SEP. This measure represents the radius of a sphere with a 50 percent confidence or probability level. This spheroid radial measure only approximates the actual 3D ellipsoid representing the uncertainties in the geocentric coordinate system. In 2D horizontal positioning, a Circular Error Probable (CEP) statistic is commonly used, particularly in military targeting. CEP represents the radius of a circle containing a 50 percent probability of position confidence.

c. Accuracy comparisons. It is important that GPS accuracy measures clearly identify the statistic from which they are derived. A "100-m" or "3-m" accuracy statistic is meaningless unless it is identified as being either 1D, 2D, or 3D, along with the applicable probability level. For example, a PPS-16 m 3D accuracy is, by definition, SEP (i.e. 50 percent). This 16-m SEP equates to 28-m 3D 95 percent confidence spheroid, or when transformed to 2D accuracy, roughly 10 m CEP, 12 m RMS, 24 m 2DRMS, and 36 m 3DRMS. See Table 5-2 for further information on GPS measurement statistics. In addition, absolute GPS point positioning accuracies are defined relative to an earth-centered coordinate system/datum. This coordinate system will differ significantly from local project or construction datums. Nominal GPS accuracies may also be published as design or tolerance limits and accuracies achieved can differ significantly from these values.

d. Dilution of Precision (DOP). The final positional accuracy of a point determined using absolute GPS survey techniques is directly related to the geometric strength of the configuration of satellites observed during the survey session. GPS errors resulting from satellite configuration geometry can be expressed in terms of DOP. In mathematical terms, DOP is a scaler quantity used in an expression of a ratio of the positioning accuracy. It is the ratio of the standard deviation of one coordinate to the measurement accuracy. DOP represents the geometrical contribution of a certain scaler factor to the uncertainty (i.e., standard deviation) of a GPS measurement. DOP values are a function of the diagonal elements of the covariance matrices of the adjusted parameters of the observed GPS signal and are used in the point formulations and determinations (Figure 5-2).

(1) General. In a more practical sense, DOP is a scaler quantity of the contribution of the configuration of satellite constellation geometry to the GPS accuracy, in other words, a measure of the "strength" of the geometry of the satellite configuration. In general, the more satellites that can be observed and used in the final solution, the better the solution. Since DOP can be used as a measure of the geometrical strength, it can also be used to selectively choose four satellites in a particular constellation that will provide the best solution.

(2) Geometric dilution of precision (GDOP). The main form of DOP used in absolute GPS positioning is the geometric DOP (GDOP), which is a measure of accuracy in a 3D position and time. The relationship between final positional accuracy, actual range error, and GDOP can be expressed as follows:

$$\sigma_a = \sigma_R * GDOP \qquad (5\text{-}9)$$

where

σ_a = final positional accuracy

σ_R = actual range error (UERE)

$$GDOP = \frac{\left[\sigma_E 2 + \sigma_N 2 + \sigma_u 2 + (c \ . \ \delta_T)^2\right]^{\frac{1}{2}}}{\sigma_R} \qquad (5\text{-}10)$$

where

σ_E = standard deviation in east value, m

σ_N = standard deviation in north value, m

σ_u = standard deviation in up direction, m

c = speed of light (299,792,458 m/s)

δ_T = standard deviation in time, s

σ_R = overall standard deviation in range, m, usually in the range of 6 m for P-code usage and 12 m for C/A-code usage

(3) Positional dilution of precision (PDOP). PDOP is a measure of the accuracy in 3D position, mathematically defined as:

$$PDOP = \frac{\left[\sigma_E 2 + \sigma_N 2 + \sigma_U 2\right]^{\frac{1}{2}}}{\sigma_R} \qquad (5\text{-}11)$$

Table 5-2
Representative GPS Error Measurement Statistics for Absolute Point Positioning

Error Measure Statistic		**Probability %**	**Relative Distance ft(σ)** (1)	**GPS Precise Positioning Service m** (2)		**GPS Standard Positioning Service m** (2)	
Linear Measures				σ_N or σ_E	σ_U	σ_N or σ_E	σ_U
Probable error		50	0.6745 σ	± 4 m	± 9 m	± 24 m	± 53 m
Average error		57.51	0.7979 σ	± 5 m	± 11 m	± 28 m	± 62 m
1-sigma standard error/deviation	(3)	68.27	1.00 σ	± 6.3 m	± 13.8 m	± 35.3 m	± 78 m
90% probability (map accuracy standard)		90	1.645 σ	± 10 m	± 23 m	± 58 m	± 128 m
95% probability/confidence		95	1.96 σ	± 12 m	± 27 m	± 69 m	± 153 m
2-sigma standard error/deviation		95.45	2.00 σ	**± 12.6 m**	**± 27.7 m**	**± 70.7 m**	**± 156 m**
99% probability/confidence		99	2.576 σ	± 16 m	± 36 m	± 91 m	± 201 m
3-sigma standard error (near certainty)		99.73	3.00 σ	± 19 m	± 42 m	± 106 m	± 234 m
Two-Dimensional Measures (4)				Circular Radius		Circular Radius	
1-sigma standard error circle (σ_c)	(5)	39	1.00 σ_c	6 m		35 m	
Circular error probable (CEP)	(6)	50	1.177 σ_c	7 m		42 m	
1-dev root mean square (1DRMS)	(3)(7)	63	1.414 σ_c	9 m		50 m	
Circular map accuracy standard		90	2.146 σ_c	13 m		76 m	
95% 2D positional confidence circle		95	2.447 σ_c	15 m		86 m	
2-dev root mean square error (2DRMS)	(8)	98^+	2.83 σ_c	**17.8 m**		**100 m**	
99% 2D positional confidence circle		99	3.035 σ_c	19 m		107 m	
3.5-sigma circular near-certainty error		99.78	3.5 σ_c	22 m		123 m	
3-dev root mean square error (3DRMS)		99.9^+	4.24 σ_c	27 m		150 m	
Three-Dimensional Measures				Spherical Radius		Spherical Radius	
1-σ spherical standard error (σ_s)	(9)	19.9	1.00 σ_s	9 m		50 m	
Spherical error probable (SEP)	(10)	50	1.54 σ_s	13.5 m		76.2 m	
Mean radial spherical error (MRSE)	(11)	61	1.73 σ_s	16 m		93 m	
90% spherical accuracy standard		90	2.50 σ_s	22 m		124 m	
95% 3D confidence spheroid		95	2.70 σ_s	24 m		134 m	
99% 3D confidence spheroid		99	3.37 σ_s	30 m		167 m	
Spherical near-certainty error		99.89	4.00 σ_s	35 m		198 m	

Notes:

Most Commonly Used Statistics Shown in Bold Face Type.
Estimates not applicable to differential GPS positioning. Circular/Spherical error radii do not have ± signs.

Absolute positional accuracies are derived from GPS simulated user range errors/deviations and resultant geocentric coordinate (X-Y-Z) solution covariance matrix, as transformed to a local datum (N-E-U or ϕ-λ-h). GPS accuracy will vary with GDOP and other numerous factors at time(s) of observation. The 3D covariance matrix yields an error ellipsoid. Transformed ellipsoidal dimensions given (i.e., σ_N- σ_E- σ_U) are only average values observed under nominal GDOP conditions. Circular (2D) and spherical (3D) radial measures are only approximations to this ellipsoid, as are probability estimates.

(Continued)

Table 5-2
(Concluded)

(1) Valid for 2-D and 3-D only if $\sigma_N = \sigma_E = \sigma_U$. ($\sigma_{min}/\sigma_{max}$) generally must be ≥ 0.2. Relative distance used unless otherwise indicated.
(2) Representative accuracy based on 1990 FRNP simulations for PPS and SPS (FRNP estimates shown in bold), and that $\sigma_N \approx \sigma_E$. SPS may have significant short-term variations from these nominal values.
(3) Statistic used to define USACE hydrographic survey depth and positioning criteria.
(4) 1990 FRNP also proposes SPS maintain, at minimum, a 2D confidence of 300 m @ 99.99% probability.
(5) $\sigma_c \approx 0.5\ (\sigma_N + \sigma_E)$ -- approximates standard error ellipse.
(6) CEP $\approx 0.589\ (\sigma_N + \sigma_E) \approx 1.18\ \sigma_c$.
(7) 1DRMS $\approx (\sigma_N^2 + \sigma_E^2)^{1/2}$.
(8) 2DRMS $\approx 2\ (\sigma_N^2 + \sigma_E^2)^{1/2}$.
(9) $\sigma_s \approx 0.333\ (\sigma_N + \sigma_E + \sigma_U)$.
(10) SEP $\approx 0.513\ (\sigma_N + \sigma_E + \sigma_U)$.
(11) MRSE $\approx (\sigma_N^2 + \sigma_E^2 + \sigma_U^2)^{1/2}$.

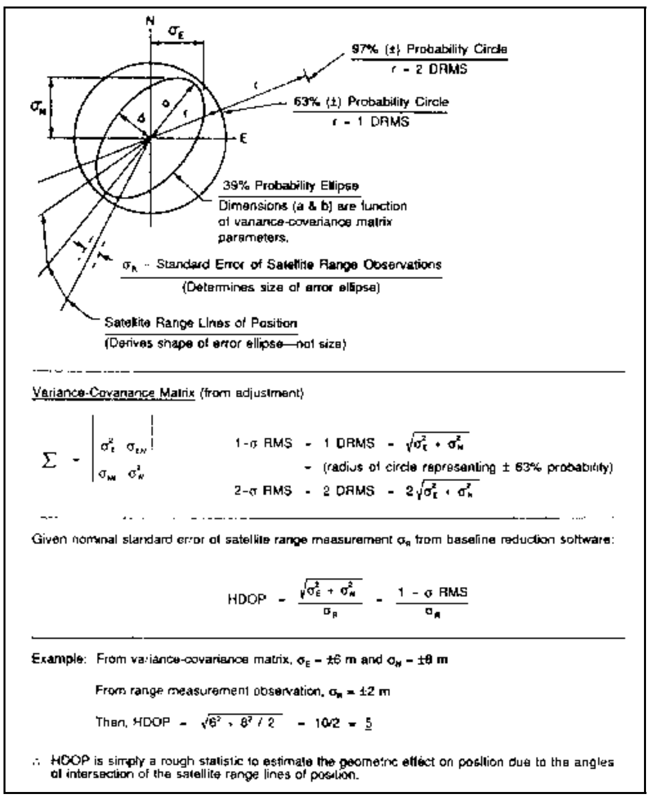

Figure 5-2. Dilution of Precision

where all variables are equivalent to those used in Equation 5-10.

(a) PDOP values are generally developed from satellite ephemerides prior to the conducting of a survey. When developed prior to a survey, PDOP can be used to determine the adequacy of a particular survey schedule. This is valid for rapid static or kinematic but is less valid for long duration static.

(b) The key to understanding PDOP is to remember that it represents position recovery at an instant in time and is not representative of a whole session of time. PDOP error is generally given in units of meters of error per 1-m error in the pseudo-range measurement (i.e., m/m). When using pseudo-range techniques, PDOP values in the range of 4-5 m/m are considered very good, while PDOP values greater than 10 m/m are considered very poor. For static surveys it is generally desirable to obtain GPS observations during a time of rapidly changing GDOP and/or PDOP.

(c) When the values of PDOP or GDOP are viewed over time, peak or high values (>10 m/m) can be associated with satellites in a constellation of poor geometry. The higher the PDOP or GDOP, the poorer the solution for that instant in time. This is critical in determining the acceptability of real-time navigation and photogrammetric solutions. Poor geometry can be the result of satellites being in the same plane, orbiting near each other, or at similar elevations.

(4) Horizontal dilution of precision (HDOP). HDOP is a measurement of the accuracy in 2D horizontal position, mathematically defined as:

$$\text{HDOP} = \frac{(\sigma_E 2 + \sigma_N 2)^{\frac{1}{2}}}{\sigma_R} \tag{5-12}$$

This HDOP statistic is most important in evaluating GPS surveys intended for horizontal control. The HDOP is basically the RMS error determined from the final variance-covariance matrix divided by the standard error of the range measurements. HDOP roughly indicates the effects of satellite range geometry on a resultant position.

(5) Vertical dilution of precision (VDOP). VDOP is a measurement of the accuracy in standard deviation in vertical height, mathematically defined as:

$$VDOP = \frac{\sigma_u}{\sigma_R} \tag{5-13}$$

(6) Acceptable DOP values. Table 5-3 indicates generally accepted DOP values for a baseline solution.

(7) Additional material. Additional material regarding GPS positional accuracy may be found in the references listed in Appendix A.

Table 5-3
Acceptable DOP Values

GDOP and PDOP: Less than 10 m/m -- optimally 4-5 m/m.

In static GPS surveying, it is desirable to have a GDOP/PDOP that changes during the time of GPS survey session.

The lower the GDOP/PDOP, the better the instantaneous point position solution is.

HDOP and VDOP: 2 m/m for the best constellation of four satellites.

Chapter 6
GPS Relative Positioning Determination Concepts

6-1. General

Absolute positioning, as discussed earlier, will not provide the accuracies needed for most USACE control projects due to existing and induced errors. In order to eliminate these errors and obtain higher accuracies, GPS can be used in a relative positioning mode. The terms “relative” and “differential” used in this chapter and throughout this manual have similar meaning. “Relative” will be used when discussing one thing in relation to another. The term “differential” will be used when discussing the technique of positioning one thing in relation to another.

6-2. Differential (Relative) Positioning

Differential or relative positioning requires at least two receivers set up at two stations (usually one is known) to collect satellite data simultaneously in order to determine coordinate differences. This method will position the two stations relative to each other (hence the term “relative positioning”) and can provide the accuracies required for basic land surveying and hydrographic surveying.

6-3. Differential Positioning (Code Pseudo-Range Tracking)

Differential positioning using code pseudo-ranges is performed similarly to that described in Chapter 5; however, some of the major uncertainties in Equations 5-1 through 5-6 are effectively eliminated or minimized. This pseudo-range process results in absolute coordinates of the user on the earth’s surface. Errors in range are directly reflected in resultant coordinate errors. Differential positioning is not so concerned with the absolute position of the user but with the relative difference between two user positions, which are simultaneously observing the same satellites. Since errors in the satellite position (X^s, Y^s, and Z^s) and atmospheric delay estimates d are effectively the same (i.e., highly correlated) at both receiving stations, they cancel each other to a large extent.

a. For example, if the true pseudo-range distance from a “known” control point to a satellite is 100 m and the observed or measured pseudo-range distance was 92 m, then the pseudo-range error or correction is 8 m for that particular satellite. A pseudo-range correction or PRC can be generated for each satellite being observed. If a second receiver is observing at least four of the same satellites and is within a reasonable distance (300 km) it can use these PRCs to obtain a relative position to the “known” control point since the errors will be similar. Thus, the relative distance (i.e., coordinate difference) between the two stations is relatively accurate (i.e., within 0.5-5 m) regardless of the poor absolute coordinates. In effect, the GPS observed baseline vectors are no different from azimuth/distance observations. As with a total station, any type of initial coordinate reference can be input to start the survey.

b. The absolute GPS coordinates will not coincide with the user’s local project datum coordinates (Figure 6-1). Since differential survey methods are concerned only with relative coordinate differences, disparities with a global reference system used by the NAVSTAR GPS are not significant for USACE purposes. Therefore, GPS coordinate differences can be applied to any type of local project reference datum (i.e., NAD 27, NAD 83, or any local project grid reference system).

c. Code pseudo-range tracking has primary application to real-time navigation systems where accuracies at the 0.5- to 5-m level are tolerable. Given these tolerances, engineering survey applications of code pseudo-range tracking GPS are limited, with two exceptions being hydrographic survey and dredge positioning. Specifications for real-time hydrographic code tracking systems are contained in EM 1110-2-1003. See Chapter 9 for further discussion on real-time code pseudo-range tracking applications.

6-4. Differential Positioning (Carrier Phase Tracking)

Differential positioning using carrier phase tracking uses a formulation of pseudo-ranges similar to those shown in Equations 5-1 through 5-6. The process becomes somewhat more complex when the carrier signals are tracked such that range changes are measured by phase resolution. In carrier phase tracking, an ambiguity factor is added to Equation 5-1 which must be resolved in order to obtain a derived range (see Figure 5-1). Methods for resolving this ambiguity (the number of unknown integer cycles) are described in Chapter 9. Carrier phase tracking provides for a more accurate range resolution due to the short wavelength (approximately 19 cm for L1 and 24 cm for L2) and the ability of a receiver to resolve the carrier phase down to about 2 mm. This method, therefore, has primary application to engineering, topographic, and geodetic surveying, and may be employed with either static

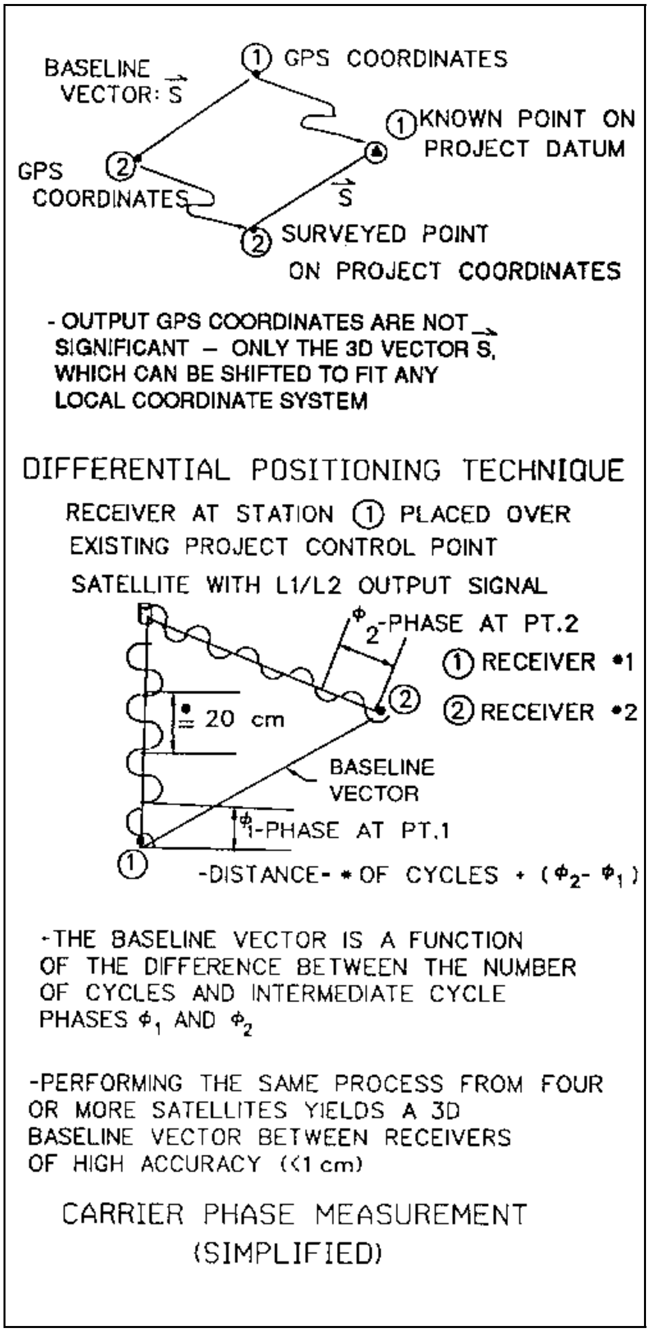

Figure 6-1. Differential positioning

or kinematic methods. There are several techniques which use the carrier phase in order to determine a station's position. These include static, rapid static, kinematic, stop and go kinematic, pseudo kinematic, and real-time kinematic (RTK) and on-the-fly (OTF) kinematic. The concepts of these techniques are explained below, but procedures can be found in Chapter 9. Table 6-1 lists these techniques, their associated accuracies, applications, and required components.

a. Static. Static surveying is the most widely used differential technique for control and geodetic surveying. It involves long observation times (1-2 hr, depending on number of visible satellites) in order to resolve the integer ambiguities between the satellite and the receiver. Accuracies in the subcentimeter range can be obtained from using the static method.

b. Rapid static. The concept of rapid static is to measure baselines and determine positions in the centimeter level with short observation times, 5-20 min. The observation time is dependent on the length of the baseline and number of visible satellites. Loss of lock, when moving from one station to the next, can also occur since each baseline is processed independent of each other.

c. Kinematic. Kinematic surveying, allows the user to rapidly and accurately measure baselines while moving from one point to the next. The data are collected and post-processed to obtain accurate positions to the centimeter level. This technique permits only partial loss of satellite lock during observation and requires a brief period of static initialization. The OTF technology, both real-time and post-processed, could eventually replace standard kinematic procedures at least for short baselines.

d. Stop and go kinematic. Stop and go kinematic involves collecting data for several minutes (1-2 min.) at each station after a period of initialization to gain the integers. This technique does not allow for loss of satellite lock during the survey. If loss of satellite lock does occur, a new period of initialization must take place. This method can be performed with two fixed or known stations in order to provide redundancy and improve accuracy.

e. Pseudo-kinematic. This technique is similar to standard kinematic procedures and static procedures combined. The differences are that there is no static initialization, longer period of time at each point (approximately 1-5 min), each point must be revisited after about an hour, and loss of satellite lock is acceptable. The positional accuracy is more than for kinematic or rapid static procedures, which makes it a less acceptable method for establishing baselines.

f. RTK and OTF carrier phase based positioning determination. The OTF/RTK positioning system uses

Table 6-1
Carrier Phase Tracking Techniques

Concept	Requirements	Applications	Accuracy
Static (Post-processing)	• L1 or L1/L2 GPS receiver • 386/486 computer for post-processing • 45 min to 1 hr minimum observation time[1]	• Control surveys (that require high accuracy)	• Subcentimeter level
Rapid Static (Post-processing)	• L1/L2 GPS receiver • 5-20 min observation time[1]	• Control surveys (that require medium to high accuracy)	• Subcentimeter level
Kinematic[2] (Post-processing)	• L1 GPS receiver with kinematic survey option • 386/486 computer for post-processing	• Continuous topo • Location surveys	• Centimeter level
Stop & Go Kinematic[2] (Post-processing)	• L1 GPS receiver • 386/486 computer for post-processing	• Medium accuracy control surveys	• Centimeter level
Pseudo Kinematic[2] (Post-processing)	• L1 GPS receiver • 386/486 computer for post-processing	• Medium accuracy control surveys	• Centimeter level
Real Time Kinematic/OTF Kinematic[3] (Real-time or post-processing)	For post-processing: • L1/L2 GPS receiver • 386/486 computer For real-time: • Internal or external processor (1- 386, 1- 486 computers w/dual com ports) • Min 4800 baud radio/modem data link set	• Real-time high accuracy hydro surveys • Location surveys • Medium accuracy control surveys • Photo control • Continuous topo	• Subdecimeter level

1. Dependent on satellite constellation and number of satellites in view.
2. Initialization period required and loss of satellite lock is not tolerated.
3. No static initialization necessary, integers gained while moving, and loss of satellite lock is tolerated.

GPS technology to allow the positioning to a subdecimeter in real time. This system determines the integer number of carrier wavelengths from the GPS antenna to the GPS satellite, transmitting them while in motion and without static initialization. The basic concept behind the OTF/RTK system is kinematic surveying without static initialization (integer initialization is performed while moving) and allows for loss of satellite lock. Other GPS techniques that can achieve this kind of accuracy require static initialization while the user is not moving and no loss of satellite lock while in motion.

6-5. Vertical Measurements with GPS

a. Elevation determination. **GPS is not recommended for Third-Order or higher vertical control surveys. It is recommended that it not be used as a substitute for standard differential leveling. It is, however, practical for small-scale topographic mapping or similar projects.**

b. Accuracy of GPS height differences. The height (h) component of GPS measurements is the weakest plane. This is due to the orbital geometry of the X-Y-Z position determination. Thus, GPS ellipsoidal height differences are usually less accurate than the horizontal components. Currently, GPS-derived elevation differences will not meet Third-Order standards as would be obtained using conventional spirit levels. Accordingly, GPS-derived elevations must be used with caution.

c. Topographic mapping with GPS. GPS positioning, whether operated in an absolute or differential positioning mode, can provide heights (or height differences) of surveyed points. The height h or height difference Δh obtained from GPS is in terms of height above or below the WGS 84 ellipsoid. These ellipsoid heights are not the same as orthometric heights, or elevations, which would be obtained from conventional differential/spirit leveling. This distinction between ellipsoid heights and orthometric elevations is critical to many engineering and construction projects; thus, users of GPS must exercise extreme caution in applying GPS height determinations to USACE projects which are based on conventional orthometric elevations.

(1) GPS uses WGS 84 as the optimal mathematical model best describing the shape of the true earth at sea level based on an ellipsoid of revolution. The WGS 84 ellipsoid adheres very well to the shape of the earth in terms of horizontal coordinates but differs somewhat with the established mean sea level definition of orthometric height. The difference between ellipsoidal height, as derived by GPS, and conventional leveled (orthometric) heights is required over an entire project area to adjust GPS heights to orthometric elevations. NGS has developed geoid modeling software (GEOID90, GEOID91, and GEOID93) to be used to convert ellipsoidal heights to approximate orthometric elevations. These values should be used with extreme caution.

(2) Static or kinematic GPS survey techniques can be used effectively on a regional basis for the densification of low-accuracy vertical control for topographic mapping purposes. Existing benchmark data (orthometric heights) and corresponding GPS-derived ellipsoidal values for at least three stations in a small project area can be used in tandem in a minimally constrained adjustment program to reasonably model the geoid. More than three correlated stations are required for larger areas to ensure proper modeling of the geoidal undulations in the area. The model from the benchmark data and corresponding GPS data can then be used to derive the unknown orthometric heights of the remaining stations occupied during the GPS observation period.

(3) Procedures for constraining GPS observations to existing vertical control are detailed in Section 11 of Leick and Lambert (1990). Step-by-step vertical control planning, observation, and adjustment procedures employed by the NGS are described in some of the publications listed in Appendix A (see Zilkoski 1990a, 1990b; Zilkoski and Hothem 1989). These procedures are recommended should a USACE field activity utilize GPS to densify low-order vertical control relative to the orthometric datum.

6-6. Differential Error Sources

The error sources encountered in the position determination using differential GPS positioning techniques are the same as those outlined in Chapter 5. In addition to these error sources, the user must ensure that the receiver maintains lock on at least three satellites for 2D positioning and four satellites for 3D positioning. When loss of lock occurs, a cycle slip (a discontinuity of an integer number of cycles in the measured carrier beat phase as recorded by the receiver) may occur. In GPS absolute surveying, if lock is not maintained, positional results will not be formulated. In GPS static surveying, if lock is not maintained, positional results may be degraded, resulting in incorrect formulations. Sometimes, in GPS static surveying, if the observation period is long enough, post-processing software may be able to average out loss of lock and cycle slips over the duration of the observation period and formulate positional results that are adequate; if this is not the case, reoccupation of the stations may be required. In all differential surveying techniques, if loss of lock does occur on some of the satellites, data processing can continue easily if a minimum of four satellites have been tracked. Generally, the more satellites tracked by the receiver, the more insensitive the receiver is to loss of lock. In general, cycle slips can be repaired.

6-7. Differential GPS Accuracies

There are two levels of accuracies obtainable from GPS using differential techniques. The first level is based on pseudo-range formulations, while the other is based on carrier beat phase formulations.

a. *Pseudo-range accuracies*. Pseudo-range formulations can be developed from either the C/A-code or the more precise P-code. Pseudo-range accuracies are generally accepted to be 1 percent of the period between successive code epochs. Use of the P-code where successive epochs are 0.1 μs apart produces results that are around 1 percent of 0.1 μs or 1 ns. Multiplying this value by the speed of light gives a theoretical resultant range measurement of around 30 cm. If using pseudo-range formulations with the C/A-code, one can expect results 10 times less precise or a range measurement precision of around 3 m. Point positioning accuracy for a differential pseudo-range formulated solution is generally found to be in the range of 0.5-10 m. These accuracies are largely dependent on the type of GPS receiver being used.

b. *Carrier beat phase formulations*. Carrier beat phase formulations can be based on either the L1 or L2, or both carrier signals. Accuracies achievable using carrier beat phase measurement are generally accepted to be 1 percent of the wavelength. Using the L1 frequency where the wavelength is around 19 cm, one can expect a theoretical resultant range measurement that is 1 percent of 19 cm, or about 2 mm. The L2 carrier can only be used with receivers which employ a cross correlation, squaring, or some other technique to get around the effects of A/S.

(1) The final positional accuracy of a point determined using differential GPS survey techniques is directly related to the geometric strength of the configuration of satellites observed during the survey session. GPS errors resulting from satellite configuration geometry can be expressed in terms of DOP. Positional accuracy for a differential carrier beat phase formulated solution is generally found to be in the range of 1-10 mm.

(2) In addition to GDOP, PDOP, HDOP, and VDOP, the quality of the baselines produced by GPS differential techniques (static or kinematic) through carrier phase recovery can be defined by a quantity called relative DOP (RDOP). Multiplying the uncertainty of a double difference measurement by RDOP yields the relative position error for that solution. Values of RDOP are measured in meters of error in relative position per error of one cycle in the phase measurement (m/cycle). Knowledge of an RDOP or a value equivalent to it is extremely important to the confidence one assigns to a baseline recovery. Key to understanding RDOP is to remember that it represents position recovery over a whole session of time and is not representative of a position recovery at an instant in time. When carrier phase recovery techniques are used, RDOP values around 0.1 m/cycle are considered acceptable.

Chapter 7
GPS Survey Equipment

7-1. GPS Receiver Selection

Selection of the right GPS receiver for a particular project is critical to its success. To ensure success, selection must be based on a sound analysis of the following criteria: applications for which the receiver is to be used, accuracy requirements, power consumption requirements, operational environment, signal processing requirements, and cost. This chapter presents only a brief overview on GPS survey equipment and selection criteria. Prior to initiating procurement, USACE Commands are advised to consult the referenced guide specifications for procuring GPS equipment.

a. *Receiver applications*. Current USACE receiver applications include land-based, water-based, and airborne applications. Land applications include surveying, geodesy, resource mapping, navigation, survey control, boundary determination, deformation monitoring, and transportation. Water or marine applications include navigation and positioning of hydrographic surveys, dredges, and drill rigs. Airborne applications include navigation and positioning of photogrammetric-based mapping. Generally, the more applications a receiver must fulfill, the more it will cost. It is important for the receiver application to be defined in order to select the proper receiver and the necessary options.

b. *Accuracy requirements*. A firm definition of the accuracy requirements (e.g., point accuracy to 100 m, 50 m, 25 m, 5 m, 1 m, cm or mm) helps to further define procedure requirements (static or kinematic), signal reception requirements (whether use of C/A- or L1/L2 P-codes is appropriate), and type of measurement required (pseudo-range or carrier beat phase measurements). This is an important part in the receiver selection process.

c. *Power requirements*. The receiver power requirements are an important factor in the determination of receiver type. Receivers currently run on a variety of power sources from A/C to 12-volt car batteries or small camcorder batteries. A high end GPS receiver can operate 3 to 4 hr on a set of batteries, whereas a low end may operate 1 to 2 days on the same set.

d. *Operational environment*. The operational environment of the survey is also an important factor in the selection of antenna type and mount, receiver dimension and weight, and durability of design. For example, the harsher the environment (high temperature and humidity variability, dirty or muddy work area, etc.), the sturdier the receiver and mount must be. The operational environment will also affect the type of power source to be used.

e. *Processing requirements*. Operational procedures required before, during, and after an observation session are very manufacturer-dependent and should be thoughtfully considered before purchase of a receiver. Often, a receiver may be easy to operate in the field, requiring very little user interface, but a tremendous amount of time and effort may be required after the survey to download the data from the receiver and process it (i.e., post-processing software may be complicated, crude, or under-developed). Also, whether a post-processed or real-time solution is desired represents a variable that is critical in determining the type of receiver to use.

f. *Cost*. Cost is a major factor in determining the type of receiver the user can purchase. Receiver hardware and software costs are a function of development costs, competition among manufacturers, and product demand. Historically, costs for the acquisition of GPS equipment have steadily fallen to the current range of prices seen today. High end receivers are upwards of $35,000 down to a low end receiver of $500.

g. *Data exchange formats*. In receiver selection it is important to remember that there is currently no standard format for exchanging data from different types of GPS receivers. However, most GPS receiver data can be put into a common text format such as RINEX. Refer to paragraph 7-4 for further discussion on receiver formats.

h. *USACE*. For most USACE civil applications, continuous tracking, C/A-code, L1 tracking, multichannel (eight or more channels) receivers are adequate. Receivers with other features may be required for a particular application. For example, a dual frequency (L1/L2) receiver with the cross correlation, squaring, or some other technique during anti-spoofing is required for the OTF and rapid static surveying techniques.

7-2. Conventional GPS Receiver Types

There are two basic types of GPS receivers: code phase and carrier phase receivers. Within these types there are C/A- and P-code receivers, codeless receivers, single- and dual-frequency receivers, and receivers that use cross correlation or squaring or P-W techniques. Figure 7-1 shows common equipment required at a station.

Figure 7-1. Common GPS equipment required at each setup

a. Code phase receivers. A code receiver is also called a “code correlating” receiver because it requires access to the satellite navigation message of the P- or C/A-code signal to function. This type of receiver relies on the satellite navigation message to provide an almanac for operation and signal processing. Because it uses the satellite navigation message, this type of receiver can produce real-time navigation data. Code receivers have “anywhere-fix” capability and, consequently, a quicker start-up time at survey commencement. An anywhere-fix receiver has the unique capability to begin calculations without being given an approximate location and time. A code receiver has anywhere-fix capability because it can synchronize itself with GPS time at a point with unknown coordinates once lock on the signals of four satellites has been obtained.

b. Carrier phase receivers. A carrier phase receiver utilizes the actual GPS signal itself to calculate a position. There are two general types of carrier phase receivers: (1) single frequency and (2) dual frequency.

(1) Single-frequency receivers. A single-frequency receiver tracks the L1 frequency signal. The single-frequency receiver generally has a lower price than the dual-frequency receiver because it has fewer components and is in greater demand. A single-frequency receiver can be used effectively to develop relative positions that are accurate over baselines of less than 50 km or where ionosphere effects can generally be ignored.

(2) Dual-frequency receivers. The dual-frequency receiver tracks both the L1 and L2 frequency signal. A dual-frequency receiver is generally more expensive than a single-frequency receiver. A dual-frequency receiver will more effectively resolve longer baselines of more than 50 km where ionosphere effects have a larger impact on calculations. Dual-frequency receivers eliminate almost all ionosphere effects by combining L1 and L2 observations. Most manufacturers of dual-frequency receivers utilize codeless techniques which allow the use of the L2 during anti-spoofing. These codeless techniques are squaring, cross-correlation, and P-W correlation.

(a) Squaring. Receivers which utilize the squaring technique are only able to obtain one-half of the signal wavelength on the L2 during anti-spoofing and have a high 30-dB loss.

(b) Cross correlation. Receivers that use this technique have a high 27-dB loss but are able to obtain the full wavelength on the L2 during anti-spoofing.

(c) P-W correlation. This method allows for both a low 14-dB loss and full wavelength on the L2 during anti-spoofing.

c. Military grade GPS receivers. The current military GPS receiver is the precise lightweight GPS receiver (PLGR), AN/PSN-11, which uses the course/acquisition (C/A), precise (P), or encrypted P(Y) codes. PLGR is designed to operate as a stand-alone unit and provide navigation information: position, velocity, and time. PLGR requires a crypto key to operate as a PPS receiver. A PPS receiver corrects for errors introduced by selective availability (S/A) and cannot be spoofed by imitated or retransmitted GPS signals, anti-spoofing (A/S). The accuracy is 16-m SEP when keyed. PLGR does not record code data because it was designed to be a navigation device, and P-code data are classified at time of reception. This also limits PLGR’s ability to be used in differential GPS. PLGR can only be used in differential GPS when using C/A code and as a rover unit. However, C/A code differential GPS is not authorized by DoD for tactical military operations. If high accuracy surveys are required during a military conflict, PPS geodetic GPS receivers are available through commercial manufacturers. PLGRs or PPS receivers are the only authorized receivers to be used in a conflict area.

(1) Non-military DoD organizations that need PLGR accuracy for their positioning requirements can purchase

PLGR from the existing DoD contract through a memorandum of agreement with DoD.

(2) Commercial GPS receiver manufacturers produce hand-held, low cost PPS GPS receivers capable of 16-m SEP accuracy when keyed. These receivers may or may not have anti-spoofing capability and require the same crypto keys as PLGR.

7-3. Receiver Manufacturers

Up-to-date listings of manufacturers are contained in various surveying trade publications. Contact should be made directly with representatives of each firm to obtain current specifications, price, availability, material, or other related data on their products.

7-4. Other Equipment

There are several other relative miscellaneous equipment items that should be considered when making a GPS receiver selection. This equipment is discussed below.

a. Data link equipment for real-time positioning. The type of data link needed for real-time positioning should be capable of transmitting digital data. The specific type of data link will depend on the user's work area and environment. Most manufacturers of GPS equipment can supply or suggest a data link that can be used for real-time positioning. Depending on the type and wattage of the data link, a frequency authorization may have to take place in order to transmit digital data. Some radio and GPS manufacturers produce 1 W or less radios for transmission of digital data which do not require frequency authorization.

b. U.S. Coast Guard (USCG) radiobeacon receivers. The USCG provides a real-time pseudo-range corrections broadcast over low frequency (270-320 kHz marine band) from a radiobeacon transmitter tower. These towers exist in most if not all coastal areas including the Mississippi River and the Great Lakes regions. The range from each tower is approximately 120 to 300 km. These corrections can be received by using a radiobeacon receiver and antenna tuned to the nearest tower site. For further information on this system contact the USCG office in your district or the number listed in Appendix C.

c. Computer equipment. Most manufacturers of GPS receivers include computer specifications needed to run their downloading and post-processing software. Most software can be run on a 386-type computer with a math co-processor or on a 486-type computer.

d. Antenna types. There are three basic types of GPS antennas: ground plane antennas, no ground plane, and choke ring antennas. Both the ground plane and the choke rings are designed to reduce the effects of multipath on the antenna.

e. Associated survey equipment. There are several accessories needed along with a GPS receiver and antenna. These include tripods, tribrachs, and tribrach adapters to name a few. Fixed height (usually 2 m) poles can be used to eliminate the need to measure antenna heights. Most of the other equipment needed is similar to what is used in a conventional survey.

7-5. GPS Common Exchange Data Format

a. RINEX. Receiver INdependent EXchange (RINEX) format is an ASCII-type format which allows a user to combine data from different manufacturer's GPS receivers. Most GPS receiver manufacturers supply programs to convert raw GPS data into a RINEX format. However, one must be careful since there are different types of RINEX conversions. Currently, the NGS distributes software which converts several receivers' raw GPS data to RINEX. NGS will distribute this software free of charge to any government agency.

b. Real-time data transmission formats. There are two types of common data formats used most often during real-time surveying: (1) RTCM SC-104 v. 2 and (2) NMEA.

(1) Transmission of data between GPS receivers. The Radio Technical Commission for Maritime Services (RTCM) is the governing body for transmissions used for maritime services. The RTCM Special Committee 104 (SC-104) has defined the format for transmission of GPS corrections. The RTCM SC-104 standard was specifically developed to address meter-level positioning requirements. This current standard transmission standard for meter-level DGPS is the RTCM SC-104 v. 2.0. This standard allows various manufacturers' equipment to work together if it is used at both the reference and remote stations. It should be noted that not all manufacturers fully support the RTCM SC-104 v. 2.0 format, and careful consideration should be made to choose one that does. A committee was formed to address the means of a transmission format for centimeter-level DGPS. This committee proposed the RTCM SC-104 v. 2.1 format, which supports raw carrier phase data, raw pseudo-range data, and corrections for both. This will allow for correction of ionosphere and troposphere errors, with dual frequency measurements, to be applied at the receiving station. It is

deemed to be downward compatible with RTCM SC-104 v. 2.0, and therefore no special transmission considerations need to be made to use it.

(2) Transmission of data between a GPS receiver and a device. The *National Maritime Electronics Association (NMEA)* governs the format of output records (i.e., the positions at the remote end). The standard concerning the corrected GPS output records at the remote receiver is referred to as the *NMEA 0183 Data Sentencing Format*. The NMEA 0183 output records can be used as input to whatever system the GPS remote receiver is interfaced. For example, GPS receivers with an NMEA 0183 output can be used to provide the positional input for a hydrographic survey system or an Electronic Chart Display and Information System (ECDIS).

Chapter 8
Planning GPS Control Surveys

8-1. General

Using differential carrier phase GPS surveying to establish control for USACE civil and military projects requires operational and procedural specifications that are a project-specific function of the control being established. To accomplish these surveys in the most efficient and cost-effective manner, and to ensure that the required accuracy criteria are obtained, a detailed survey planning phase is essential. This chapter defines GPS survey design criteria and related observing specifications required to establish control for USACE military construction and civil works projects. Information on cost for GPS surveys can be found in Chapter 12, and information on using GPS for hydrographic surveys can be found in EM 1110-2-1003.

8-2. Required Project Control Accuracy

The first step in planning GPS control surveys is to determine the ultimate accuracy requirements. Survey accuracy requirements are a direct function of specific project functional needs, that is, the basic requirements needed to support planning, engineering design, maintenance, operations, construction, or real estate. This is true regardless of whether GPS or conventional surveying methods are employed to establish project control. Most USACE military and civil works engineering/construction activities require relative accuracies (i.e., accuracies between adjacent control points) ranging from 1:1,000 to 1:50,000, depending on the nature and scope of the project. Few USACE projects demand relative positional accuracies higher than the 1:50,000 level (Second-Order, Class I). Since the advent of GPS survey technology, there has been a tendency to specify higher accuracies than necessary. Specifying higher accuracy levels than those minimally required for the project can unnecessarily increase project costs.

a. Project functional requirements. Project functional requirements must include planned and future design, construction, and mapping activities. Specific control density and accuracy are designed from these functional requirements.

(1) Density of control within a given project is determined from factors such as planned construction, site plan mapping scales, master plan mapping scale, and dredging and hydrographic survey positioning requirements.

(2) The relative accuracy for project control is also determined based on mapping scales, design/construction needs, type of project, etc. Most site plan mapping for design purposes is performed and evaluated relative to American Society of Photogrammetry and Remote Sensing (ASPRS) standards. These standards apply to photogrammetric mapping, plane table mapping, total station mapping, etc. Network control must be of sufficient relative accuracy to enable hired-labor or contracted survey forces to reliably connect their supplemental mapping work.

b. Minimum accuracy requirements. Project control surveys shall be planned, designed, and executed to achieve the minimum accuracy demanded by the project's functional requirements. In order to most efficiently utilize USACE resources, control surveys shall not be designed or performed to achieve accuracy levels that exceed the project requirements. For instance, if a Third-Order, Class I accuracy standard (1:10,000) is required for offshore dredge/survey control on a navigation project, field survey criteria shall be designed to meet this minimum standard.

c. Achievable GPS accuracy. As stated previously, GPS survey methods are capable of providing significantly higher relative positional accuracies with only minimal field observations, as compared with conventional triangulation, trilateration, or EDM traverse. Although a GPS survey may be designed and performed to support lower accuracy project control requirements, the actual results could generally be several magnitudes better than the requirement. Although higher accuracy levels are relatively easily achievable with GPS, it is important to consider the ultimate use of the control on the project in planning and designing GPS control networks. Thus, GPS survey adequacy evaluations should be based on the project accuracy standards, not those theoretically obtainable with GPS.

(1) For instance, an adjustment of a pair of GPS-established points may indicate a relative distance accuracy of 1:800,000 between them. These two points may be subsequently used to set a dredging baseline using 1:2,500 construction survey methods; and from 100-ft-spaced stations on this baseline, cross sections are projected using 1:500 to 1:1,000 relative accuracy methods (typical hydrographic surveys). Had the GPS-observed baseline been accurate only to 1:20,000, such a closure would still have easily met the project's functional requirements.

(2) Likewise, in plane table topographic (site plan) mapping or photogrammetric mapping work, the difference between 1:20,000 and 1:800,000 relative accuracies is not perceptible at typical USACE mapping/construction scales (1:240 to 1:6,000), or ensuring supplemental compliance with ASPRS standards. In all cases of planimetric and topographic mapping work, the primary control network shall be of sufficient accuracy such that ASPRS standards can be met when site plan mapping data are derived from such points. For most large-scale military and civil mapping work performed by USACE, Third-Order relative accuracies are adequate to control planimetric and topographic features within the extent of a given sheet/map or construction site. On some projects covering large geographical areas (e.g., reservoirs, levee systems, installations), this Third-Order mapping control may need to be connected to/with a Second-Order (Class I or II) network to minimize scale distortions over longer reaches of the project.

(3) In densifying control for GIS databases, the functional accuracy of the GIS database must be kept in perspective with the survey control requirements. Performing 1:100,000 accuracy surveys for a GIS level containing 1-acre cell definitions would not be cost-effective; sufficient accuracy could be obtained by scaling relative coordinates from a U.S. Geological Survey (USGS) quadrangle map.

8-3. General GPS Network Design Factors

Some, but not all, of the factors to be considered in designing a GPS network (and subsequent observing procedure) should include the following:

a. Project size. The extent of the project will affect the GPS survey network shape. Many civil works navigation and flood control projects are relatively narrow in lateral extent but may extend for many miles longitudinally. Alternatively, military installations or reservoir/recreation projects may project equally in length and breadth. The optimum GPS survey design will vary considerably for these different conditions.

b. Required density of control. The type of GPS survey scheme used will depend on the number and spacing of points to be established, which is a project-specific requirement. In addition, maximum baseline lengths between stations and/or existing control are also prescribed. Often, a combination of GPS and conventional survey densification will prove to be the most cost-effective approach.

c. Absolute GPS reference datums. Coordinate data for GPS baseline observations are referenced and reduced relative to WGS 84, an earth-centered (geocentric) coordinate system. This system is not directly referenced to but is closely related to, for all practical purposes, GRS 80 upon which North American Datum of 1983 (NAD 83) is related (for CONUS work). GPS data reduction and adjustment are normally performed using the WGS 84 earth-centered (geocentric) coordinate system (X-Y-Z), with baseline vector components (ΔX, ΔY, ΔZ) measured relative to this coordinate system. Although baseline vectors are measured relative to the WGS 84 system, for most USACE engineering and construction applications these data may be used in adjustments on NAD 27 (Clarke 1866). (See paragraphs 3-4 and 4-1.)

(1) If the external network being connected (and adjusted to) is the published NAD 83, the GPS baseline coordinates may be directly referenced on the GRS 80 ellipsoid since they are nearly equal. All supplemental control established is therefore referenced to the GRS 80/NAD 83 coordinate system.

(2) If a GPS survey is connected to NAD 27 (SPCS 27) stations which were not adjusted to the NAD 83 datum, then these fixed points may be transformed to NAD 83 coordinates using USACE program CORPSCON (see EM 1110-1-1004) and the baseline reductions and adjustment performed relative to the GRS 80 ellipsoid. This method is recommended for USACE projects, only if resurveying is not a viable option.

(3) Alternatively, GPS baseline connections to NAD 27 (SPCS 27) project control may be reduced and adjusted directly on that datum with resultant coordinates on the NAD 27. Geocentric coordinates on the NAD 27 datum may be computed using the transformation algorithms given in Chapter 11. Refer also to EM 1110-1-1004 regarding state plane coordinate transforms between SPCS 27 and SPCS 83 grids. Conversions of final adjusted points on the NAD 27 datum to NAD 83 may also be performed using CORPSCON.

(4) Ellipsoid heights h referenced to the GRS 80 ellipsoid differ significantly from the orthometric elevations H on NGVD 29, NAVD 88, or dynamic/hydraulic elevations on the IGLD 55, IGLD 85. This difference (geoid separation, or N) can usually be ignored for horizontal control. This implies N is assumed to be zero and $h = H$ where the elevation may be measured, estimated, or scaled at the fixed point(s). See Chapter 6 for using GPS for vertical surveys.

(5) Datum systems other than NAD 27/NAD 83 will be used in OCONUS locations. Selected military operational requirements in CONUS may also require non-NAD datum references. It is recommended that GPS baselines be directly adjusted on the specific project datum.

d. Connections to existing control. For most static and kinematic GPS horizontal control work, at least two existing control points should be connected for referencing and adjusting a new GPS survey (Table 8-1). Existing points may be part of the NGRS or in-place project control which has been adequately used for years. Additional points may be connected if practical. In some instances, a single existing point may be used to generate spurred baseline vectors for supplemental construction control.

(1) Connections with existing project control. The first choice for referencing new GPS surveys is the existing project control. This is true for most surveying, not just GPS, and has considerable legal basis. Unless a newly authorized project is involved, long-established project control reference points should be used. If the project is currently on a local datum, then a supplemental tie to the NGRS should be considered as part of the project.

(2) Connections with NGRS. Connections with the NGRS (i.e., National Ocean Service/National Geodetic Survey control on NAD 83) are preferred where prudent and practical. As with conventional USACE surveying, such connections to the NGRS are not mandatory. In many instances, connections with the NGRS are difficult and may add undue cost to a project with limited resources. When existing project control is known to be of poor accuracy, then ties (and total readjustment) to the NGRS may be warranted. Sufficient project funds should have been programmed to cover the additional costs of these connections, including data submittal and review efforts if such work is intended to be included in the NGRS. (See paragraph 1-8*c* regarding advance programming requirements.)

(3) Mixed NGRS and project control connections. On existing projects, NGRS-referenced points should not be mixed with existing project control. This is especially important if existing project control was poorly connected with the older (NAD 27) NGRS, or if the method of this original connection is uncertain. Since NGRS control has been readjusted to NAD 83 (including subsequent high-precision HARNS readjustments of NAD 83) and most USACE project control has not, problems may result if these schemes are mixed indiscriminately. If a decision is made to establish and/or update control on an existing project, and connections with the NGRS (NAD 83/86) are required, then all existing project control points must be resurveyed and readjusted. Mixing different reference systems can result in different datums, with obvious adverse impacts on subsequent construction or boundary reference. It is far preferable to use “weak” existing (long-established) project control (on NAD 27 or whatever datum) for reference than to end up with a mixture of different systems or datums. See EM 1110-1-1004 for further discussion.

(4) Accuracy of connected reference control. Ideally, connections should be made to control of a higher order of accuracy than that intended for the project. In cases where NGRS control is readily available, this is usually the case. However, when only existing project control is available, connection and adjustment will have to be performed using that reference system, regardless of its accuracy. GPS baseline measurements should be performed over existing control to assess its accuracy and adequacy for adjustment, or to configure partially constrained adjustments.

(5) Connection constraints. Although Table 8-1 requires only a minimum of two existing stations to reliably connect GPS static and kinematic surveys, it may often be prudent to include additional NGRS and/or project points, especially if the existing network is of poor reliability. Adding additional points provides redundant checks on the surrounding network. This allows for the elimination of these points should the final constrained adjustment indicate a problem with one or more of the fixed points.

(a) Table 8-1 indicates the maximum allowable distance from the existing network that GPS baselines should extend.

(b) Federal Geodetic Control Subcommittee (FGCS) GPS standards (FGCC 1988) require connections to be spread over different quadrants relative to the survey project. Other GPS standards suggest an equilateral distribution of fixed control about the proposed survey area. These requirements are unnecessary for USACE work. The value shown in Table 8-1 (for Second-Order, Class I) is only suggested and not mandatory.

e. Location feasibility and field reconnaissance. A good advance reconnaissance of all marks within the project is crucial to the expedient and successful

Table 8-1
GPS Survey Design, Geometry, Connection, and Observing Criteria

	Classification Order			
Criterion	2nd, **I**	2nd, **II**	3rd, **I**	3rd, **II**
Relative accuracy				
ppm	20	50	100	200
1 part in	50k	20k	10k	5k
Required connections to existing horizontal control				
NGRS network		W/F/P		
Local project network		Yes		
Baseline observation check required over existing control	Yes	W/F/P	W/F/P	No
Number of connections with existing network (NGRS or local project control)				
Minimum	2	2	2	2
Optimum	3	3	2	2
New point spacing, m, not less than	1,000	500	200	100
Maximum distance from network to nearest control point in project, km	50	50	50	50
Minimum network control quadrant location (relative to project center)	2	N/R	N/R	N/R
Multiple station occupations (static GPS surveys)				
% Occupied three times	N/R	N/R	N/R	N/R
% Occupied two times	N/R	N/R	N/R	N/R
Repeat baseline observations (% of total baselines)	0	0	0	0
Master or fiducial stations required	W/F/P	No	No	No
Loop closure requirements:				
Maximum number of baselines/loop	10	20	20	20
Maximum loop length, km, not to exceed	100	200	N/R	N/R
Loop misclosure, ppm, not less than	20	50	100	200
Single spur baseline observations				
Allowed per order/class	No	No	Yes	Yes
Number of sessions/baseline	-	-	2	2
Required tie to NGRS	-	-	No	No
Field observing criteria -- static GPS surveys				
Required antenna phase height measurement per session	2	2	2	2
Meteorological observations required	No	No	No	No
Two frequency L1/L2 observations required:				
< 50-km lines	No	No	No	No
> 50-km lines	Yes	Yes	Yes	Yes

(Continued)

Table 8-1
(Concluded)

Criterion	Classification Order 2nd, **I**	2nd, **II**	3rd, **I**	3rd, **II**
Recommended minimum observing time (per session), min	60	45	30	30
Minimum number of sessions per GPS baseline	1	1	1	1
Satellite quadrants observed (minimum number)	3 W/F/P	N/R	N/R	N/R
Minimum obstruction angle above horizon, deg	15	15	15	15
Maximum HDOP/VDOP during session	N/R	N/R	N/R	N/R
Photograph and/or pencil rubbing required	A/R	No	No	No
Kinematic GPS surveying				
Allowable per survey class	Yes	Yes	Yes	Yes
Required tie to NGRS	W/F/P	W/F/P	No	No
Measurement time/baseline, min	(follow manufacturer's specifications) A/R			
Minimum number of reference points:	2	2	1-2	1
Preferred references	2	2	2	1
Maximum PDOP		15		
Minimum number of observations from each reference station	2	2	2	2
Adjustment and data submittal requirements				
Approximate adjustments allowed	Yes	Yes	Yes	Yes
Contract acceptance criteria				
Type of adjustment	Free (unconstrained)			
Evaluation statistic	Relative distance accuracies			
Error ellipse sizes	(not used as criteria)			
Histograms	(not used as criteria)			
Reject criteria				
Statistic	Normalized residual			
Standard	±3 * SEUW			
Optimum/nominal weighting				
Horizontal	± 5 + 2 ppm			
Vertical	± 10 + 2 ppm			
Optimum variance of unit weight	between 0.5 and 1.5			
GPS station/session data recording format	Bound field survey book or form			
Final station descriptions	Standard DA form			
FGCS/NGS Bluebook required	No			
Written project/adjustment report required	Yes			

Notes:

1. Abbreviations used in this table are explained as follows:
   W/F/P--Where feasible and practical.
   N/R--No requirement for this specification--usually indicates variance with provisional FGCC GPS specifications.
   A/R--As required in specific project instructions or manufacturer's operating manual.
   SEUW--Standard error of unit weight.
2. Classification orders refer to intended survey precision for USACE application, not necessarily FGCC standards designed to support national network densification.

completion of a GPS survey. The site reconnaissance should ideally be completed before the survey is started. The surveyor should also prepare a site sketch and brief description on how to reach the point since the individual performing the site reconnaissance may not be the surveyor that returns to occupy the known or unknown station.

(1) Project sketch. A project sketch should be developed before any site reconnaissance is performed. The sketch should be on a 7-1/2-min USGS quadrangle map or other suitable drawing. Drawing the sketch on the map will assist the planner in determination of site selections and travel distances between stations.

(2) Station descriptions and recovery notes. Station descriptions for all new monuments will be developed as the monumentation is performed. The format of these descriptions will follow that stated in EM 1110-1-1002. Recovery notes should be written for existing NGRS network stations and project control points, as detailed in EM 1110-1-1002. Estimated travel times to all stations should be included in the description. Include road travel time, walking time, and GPS receiver breakdown and setup time. These times can be estimated while performing the initial reconnaissance. A site sketch shall also be made on the description/recovery form. Examples of site reconnaissance reports are shown in Figures 8-1 and 8-2. A blank reconnaissance report form is included as Worksheet 8-1 (Figure 8-3), which may be used in lieu of a standard field survey book.

(3) Way point navigation. Way point navigation is an option on some receivers, allowing the user to enter geodetic position (usually latitude and longitude) of points of interest along a particular route the user may wish to follow. The GPS antenna, fastened to a vehicle or range pole, and receiver can then provide the user with navigational information. The navigational information may include the distance and bearing to the point of destination (stored in the receiver), the estimated time to destination, and the speed and course of the user. The resultant message produced can then be used to guide the user to the point of interest. Way point navigation is an option that, besides providing navigation information, may be helpful in the recovery of control stations which do not have descriptions. If the user has the capability of real-time code phase positioning, the way point navigational accuracy can be in the range of 0.5 - 10 m.

(4) Site obstruction/visibility sketches. The individual performing the site reconnaissance should record the azimuth and vertical angle of all obstructions. The azimuths and vertical angles should be determined with a compass and inclinometer. Because obstructions such as trees and buildings cause the GPS signal transmitted from the GPS satellite to be blocked, the type of obstruction is also an important item to be recorded, see Figure 8-2. The type of obstruction is also important to determine if multipath might cause a problem. Multipath is caused by the reflection of the GPS signal by a nearby object producing a false signal at the GPS antenna. Buildings with reflective surfaces, chain-link fences, and antenna arrays are objects that may cause multipath. The site obstruction data are needed to determine if the survey site is suitable for GPS surveying. Obstruction data should be plotted on a Station Visibility Diagram, as shown in Figure 8-4. (A blank copy of this form is provided as Worksheet 8-2 (Figure 8-5).) GPS surveying does require that all stations have an unobstructed view 15 deg above the horizon, and satellites below 10 deg should not be observed.

(5) Suitability for kinematic observations. Clear, obstruction-free projects may be suitable for kinematic GPS surveys as opposed to static. The use of kinematic observations will increase productivity by a factor of 5 to 10 over static methods, while still providing adequate accuracy levels. On many projects, a mixture between both static and kinematic GPS observations may prove to be most cost-effective.

(6) Monumentation. All monumentation should follow the guidelines of EM 1110-1-1002.

(7) On-site physical restrictions. The degree of difficulty in occupying points due to such factors as travel times, site access, multipath effects, and satellite visibility should be anticipated. The need for redundant observations, should reobservations be required, must also be considered.

(8) Checks for disturbed existing control. Additional GPS baselines may need to be observed between existing NGRS/project control to verify their accuracy and/or stability.

(9) Satellite visibility limitations. For most of the Continental United States, there are at least four to five satellites in view at all times. However, some areas may have less during times of satellite maintenance or unhealthy satellites. Satellite visibility charts of the GPS satellite constellation play a major part in optimizing network configurations and observation schedules.

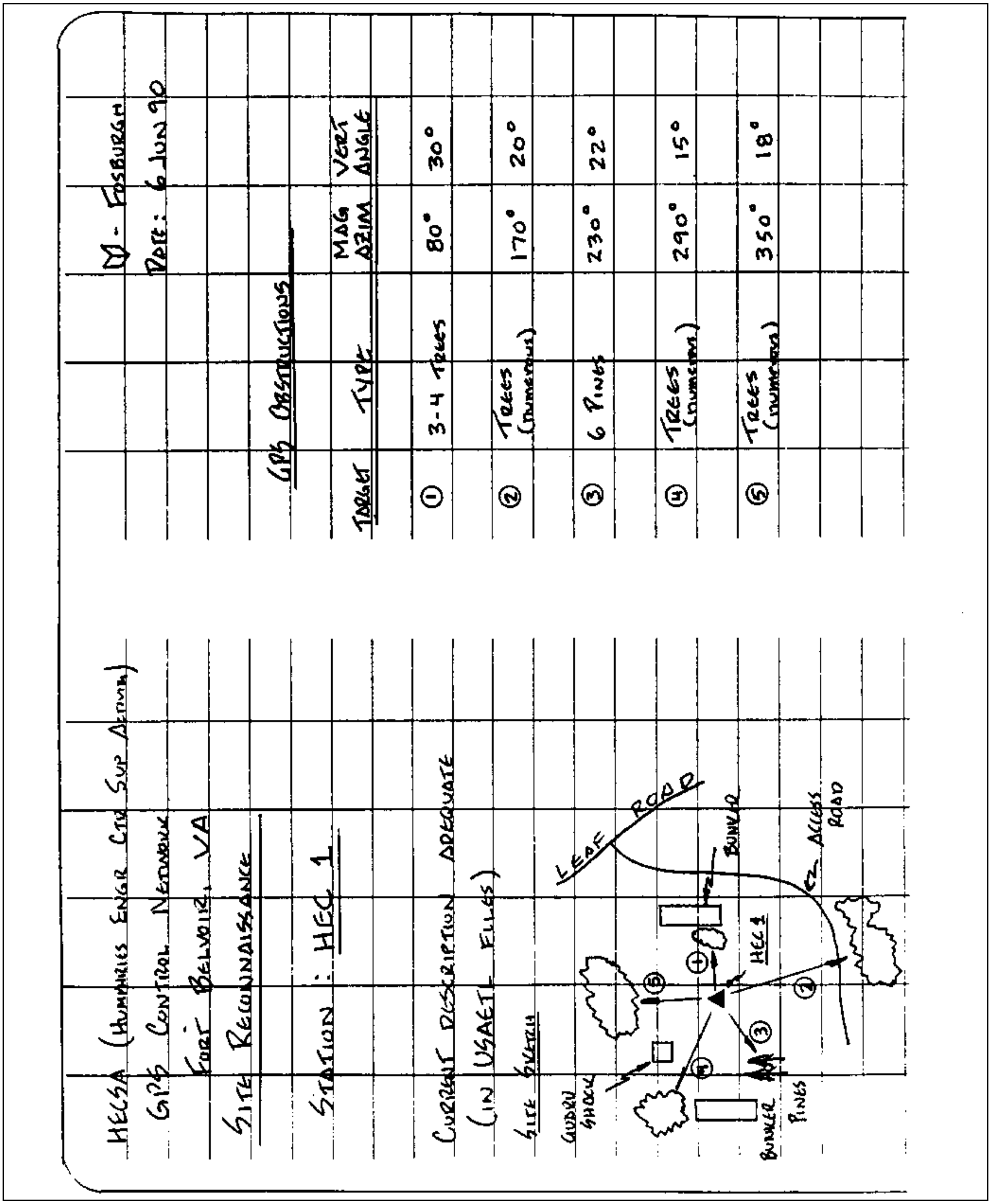

Figure 8-1. Sample site reconnaissance sketch

SITE RECONNAISSANCE/REPORT ON CONDITION OF SURVEY MARK

Project for Which Reconnaissance was Performed DWORSHAK DAM

Station Name OROFINO   Year Established 1933

State Code ID   County POTTER   Map Scale 1:24,000

Organization's Mark C&GS   Map Sheet CLEARWATER

Search Performed By K. SMITH   Date 4/12/89

Organization WALLA WALLA DISTRICT

Exact Stamping OROFINO 1933   Condition GOOD

Please report on the thoroughness of the search in case the mark was not recovered. Suggest changes in description, need for repairing or moving the mark, or other pertinent facts. Record letters and numbers found stamped in (not cast in) the mark.

THE MARK WAS RECOVERED USING THE 1970
DESCRIPTION. ADDITIONAL DESCRIPTIVED DATA:
THE MARK IS 89.7' W OF PP#6342, 62.4' NE OF AN 18"
MAPLE, 42.0' S OF A 10" SPRUCE AND 2' E OF AN ORANGE
WITNESS POST.
RECOVERED REFERENCE MARK OROFINO No. 1 1933 GOOD
" " " OROFINO No. 3 1970 GOOD

****

TRAVEL TIME BY 2-WHEEL DRIVE VEHICLE FROM CLEARWATER IS APPROX. 15 MINUTES.

SKETCH

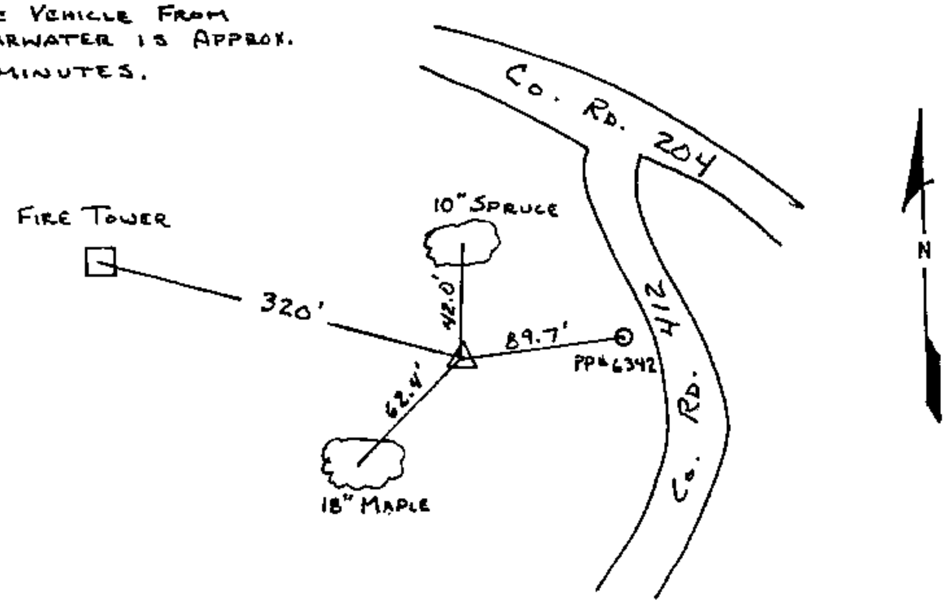

Figure 8-2. Reconnaissance report on condition of survey

SITE RECONNAISSANCE/REPORT ON CONDITION OF SURVEY MARK

Project for Which Reconnaissance was Performed ____________________

Station Name ____________________ Year Established ________

State Code ____ County ____________________ Map Scale ____________

Organization's Mark ____________________ Map Sheet ____________

Search Performed By ____________________ Date ____________

Organization ____

Exact Stamping ____________________ Condition ____________

Please report on the thoroughness of the search in case the mark was not recovered. Suggest changes in description, need for repairing or moving the mark, or other pertinent facts. Record letters and numbers found stamped in (not cast in) the mark.

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SKETCH

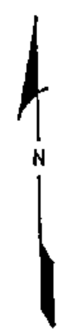
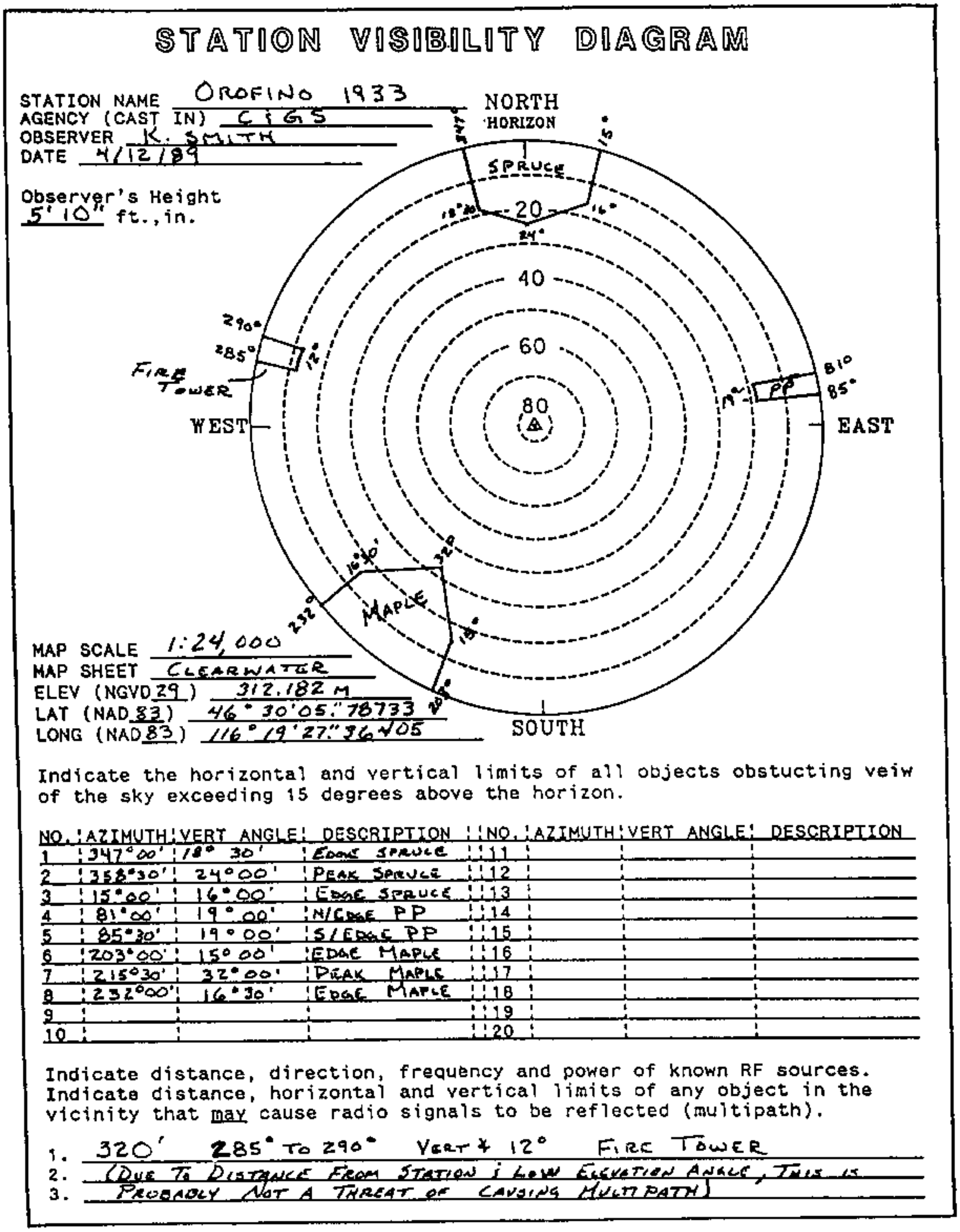


Figure 8-3. Worksheet 8-1, Site Reconnaissance Report form



STATION VISIBILITY DIAGRAM

STATION NAME Orofino 1933
AGENCY (CAST IN) C & GS
OBSERVER K. Smith
DATE 4/12/89

Observer's Height
5' 10" ft.,in.

MAP SCALE 1:24,000
MAP SHEET Clearwater
ELEV (NGVD 29) 312.182 m
LAT (NAD 83) 46° 30'05."78733 N
LONG (NAD 83) 116° 19'27."36405 W

Indicate the horizontal and vertical limits of all objects obstucting veiw of the sky exceeding 15 degrees above the horizon.

NO.	AZIMUTH	VERT ANGLE	DESCRIPTION	NO.	AZIMUTH	VERT ANGLE	DESCRIPTION
1	347°00'	18° 30'	Edge Spruce	11			
2	358°30'	24°00'	Peak Spruce	12			
3	15°00'	16°00'	Edge Spruce	13			
4	81°00'	19°00'	N/Edge PP	14			
5	85°30'	19°00'	S/Edge PP	15			
6	203°00'	15°00'	Edge Maple	16			
7	215°30'	32°00'	Peak Maple	17			
8	232°00'	16°30'	Edge Maple	18			
9				19			
10				20			

Indicate distance, direction, frequency and power of known RF sources. Indicate distance, horizontal and vertical limits of any object in the vicinity that may cause radio signals to be reflected (multipath).

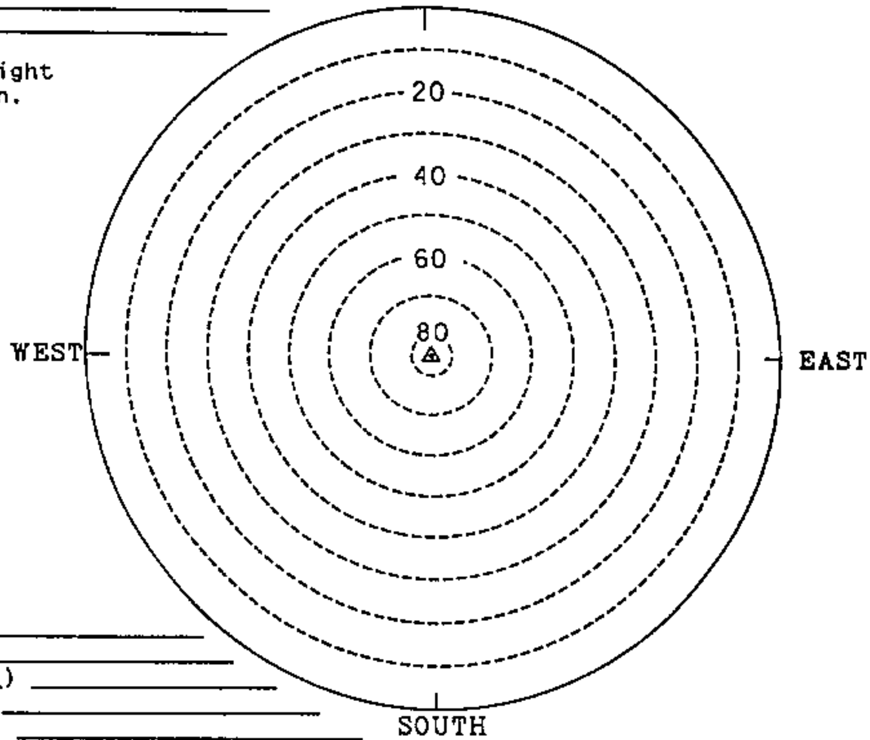
1. 320' 285° to 290° Vert ∡ 12° Fire Tower
2. (Due to Distance From Station & Low Elevation Angle, This is
3. Probably Not a Threat of Causing Multipath)

Figure 8-4. Sample station visibility diagram

STATION VISIBILITY DIAGRAM

STATION NAME ____________________
AGENCY (CAST IN) ____________________
OBSERVER ____________________
DATE ____________________

Observer's Height
________ ft.,in.



MAP SCALE ____________________
MAP SHEET ____________________
ELEV (NGVD___) ____________________
LAT (NAD___) ____________________
LONG (NAD___) ____________________

Indicate the horizontal and vertical limits of all objects obstucting veiw of the sky exceeding 15 degrees above the horizon.

NO.	AZIMUTH	VERT ANGLE	DESCRIPTION	NO.	AZIMUTH	VERT ANGLE	DESCRIPTION
1				11			
2				12			
3				13			
4				14			
5				15			
6				16			
7				17			
8				18			
9				19			
10				20			

Indicate distance, direction, frequency, and power of known RF sources. Indicate distance, horizontal and vertical limits of any object in the vicinity that <u>may</u> cause radio signals to be reflected (multipath).

1. ____________________
2. ____________________
3. ____________________

Figure 8-5. Worksheet 8-2, Station Visibility Diagram

(10) Station intervisibility requirements. Project specifications may dictate station intervisibility for azimuth reference. This may constrain minimum station spacing.

f. Multiple/repeat baseline connections. Table 8-1 lists recommended criteria for baseline connections between stations, repeat baseline observations, and multiple station occupations. Many of these standards were developed by FGCS for performing high-precision geodetic control surveys such that extensive redundancy will result from the collected data. Since the purpose of these geodetic densification surveys is markedly different from USACE control densification, the need for such high observational redundancy is also different. Adding redundant baseline/station occupations may prove prudent on some remote projects where accessibility is difficult.

g. Loop requirements. Loops (i.e., traverses) provide the mechanism for performing field data validation as well as final adjustment accuracy analysis. Since loops of GPS baselines are comparable to traditional EDM/taped traverse routes, misclosures and adjustments can be handled similarly. Most GPS survey nets (static or kinematic) end up with one or more interconnecting loops that are either internal from a single fixed point or external through two or more fixed network points. Loops should be closed off at the spacing indicated in Table 8-1. Loop closures should meet the criteria specified in Table 8-1, based on the total loop length. See also Chapter 10 for additional GPS loop closure checks.

(1) GPS control surveys may be conducted by forming loops between two or more existing points, with adequate cross-connections where feasible. Such alignment procedures are usually most practical on civil works navigation projects which typically require control to be established along a linear path, e.g., river or canal embankments, levees, beach renourishment projects, and jetties.

(2) Loops should be formed every 10 to 20 baselines, preferably closing on existing control.

(3) Connections to existing control should be made as opportunities exist and/or as often as practical.

(4) When establishing control over relatively large military installations, civil recreation projects, flood control projects, and the like, a series of redundant baselines forming interconnecting loops is usually recommended. When densifying Second- and Third-Order control for site plan design and construction, extensive cross-connecting loop and network configurations recommended by the FGCS for geodetic surveying are not necessary.

(5) On all projects, maximum use of combined static and kinematic GPS observations should be considered, both of which may be configured to form pseudo-traverse loops for subsequent field data validation and final adjustment.

8-4. GPS Network Design and Layout

A wide variety of survey configuration methods may be used to densify project control using GPS survey techniques. Unlike conventional triangulation, trilateration, and EDM traverse surveying, the shape, or geometry, of the GPS network design is not as significant. The following guidelines for planning and designing proposed GPS surveys are intended to support lower order (Second-Order, Class I, or 1:50,000 or less accuracy) control surveys applicable to USACE civil works and military construction activities. An exception to this would be GPS surveys supporting structural deformation monitoring projects where relative accuracies at the centimeter level or better are required over a small project area.

a. Newly established GPS control may or may not be incorporated into the NGRS, depending on the adequacy of the connection to the existing NGRS network, or whether it was tied only internally to existing project control.

b. Of paramount importance in developing a network design is to obtain the most economical coverage within the prescribed project accuracy requirements. The optimum network design, therefore, provides a minimum amount of baseline/loop redundancy without an unnecessary amount of “over-observation.” Obtaining this optimum design (cost versus accuracy) is difficult and constantly changing due to evolving GPS technology and satellite coverage.

c. GPS survey layout schemes. The planning of a GPS survey scheme is similar to that for conventional triangulation or traversing. The type of survey design adopted is dependent on the GPS technique employed and the requirements of the user.

(1) GPS networking. A GPS network is proposed when established survey control is to be used in precise network densification (1:50,000-1:100,000). For lower order work (i.e., less than 1:50,000), elaborate network schemes are unnecessary and less work-intensive GPS

survey extension methods may be used. When the networking method is selected, the surveyor should devise a survey network that is geometrically sound. Triangles that are weak geometrically should be avoided. The networking method is practical only with static, pseudo-kinematic, and kinematic survey techniques. Figure 8-6 shows an example of a step-by-step method to build a GPS survey network.

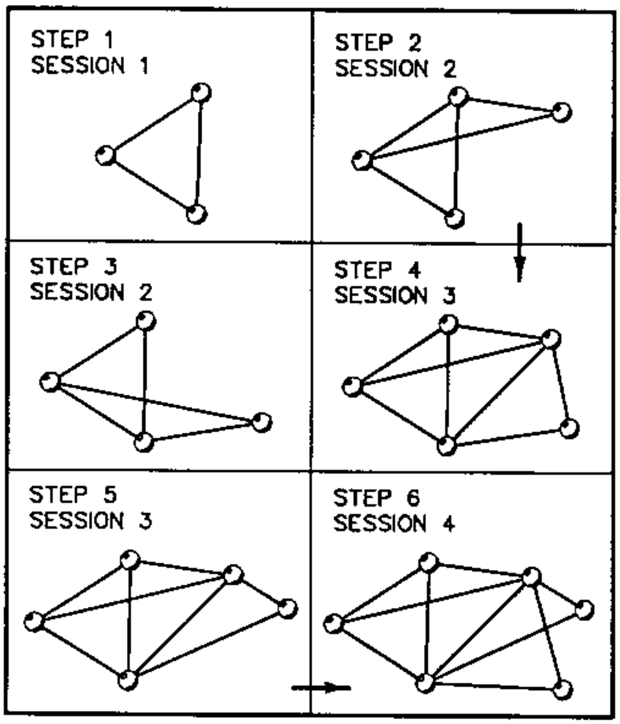


Figure 8-6. GPS network design

(2) GPS traversing. Traversing is the method of choice when the user has only two or three receivers and required accuracies are 1:5,000-1:50,000. Traversing with GPS is done similar to conventional methods. Open-end traverses are not recommended when 1:5,000 accuracies or greater are required. Since GPS does not provide sufficient point positioning accuracies, the surveyor must have a minimum of one fixed (or known) control point, although three are preferred. A fixed control point is a station with known latitude-longitude-height or easting-northing-height. This point may or may not be part of the NGRS. If only one control point is used and the station does not have a known height, the user will be unable to position the unknown stations. When performing a loop traverse, the surveyor should observe a check angle or check azimuth using conventional survey techniques to determine if the known station has been disturbed. If azimuth targets are not visible, and a check angle cannot be observed, a closed traverse involving one or more control points is recommended. Again, a check angle or check azimuth should be observed from the starting control station. If a check angle is not performed, the survey can still be completed. However, if the survey does not meet specified closure requirements, the surveyor will be unable to assess what control point may be in error. If a check angle or check azimuth cannot be observed, a third control point should be tied into the traverse (Figure 8-7). This will aid in determining the cause of misclosure.

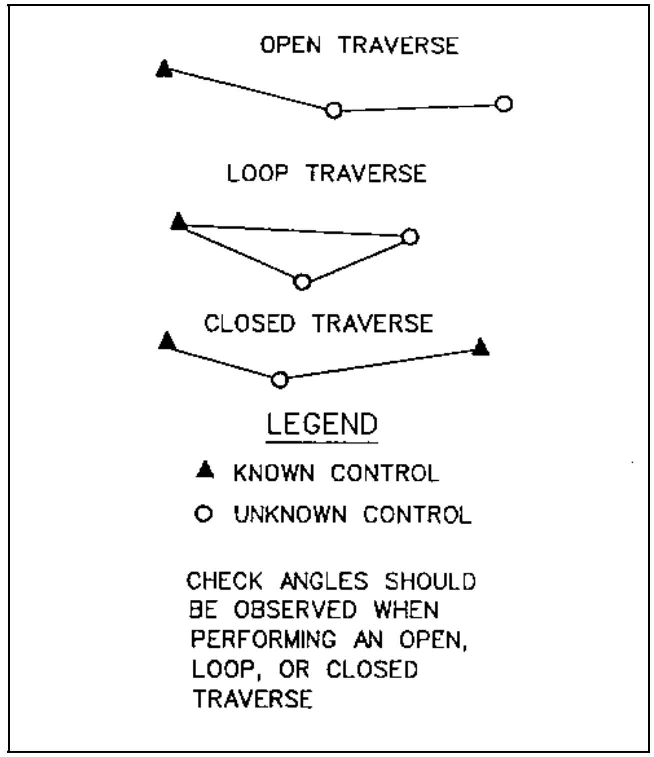


Figure 8-7. GPS traversing schemes

(3) GPS spur shots. Spurs are an acceptable method when the user has only two receivers or only a few control points are to be established. Spur lines should be observed twice during two independent observing sessions. Once the first session is completed, the receivers at each station should be turned off and the tripod elevations changed. This procedure is similar to performing a forward and backward level line. It is important that the tripods be moved in elevation and replumbed over the control station between sessions. If this step is not implemented, the two baselines cannot be considered independent. Figure 8-8 shows an example of a spur line. Spurs are most applicable to static survey and relative positioning (code phase) techniques.

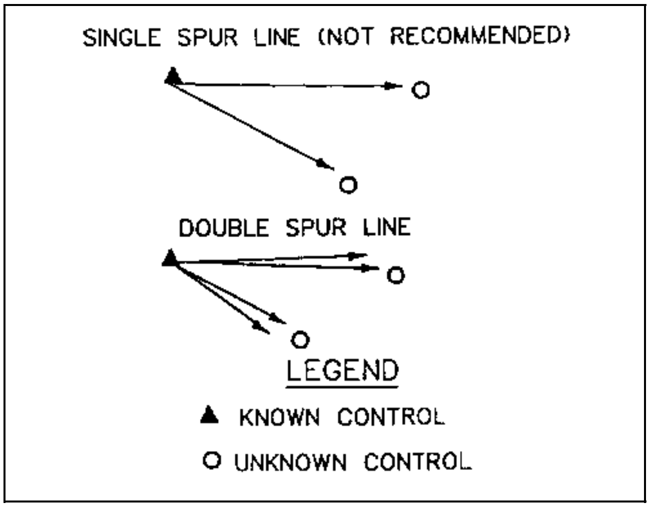


Figure 8-8. GPS spur line

8-5. GPS Techniques Needed for Survey

After a GPS network has been designed and laid out, a GPS survey method or technique needs to be considered. The concepts for each method were discussed in Chapter 6 and the procedures are discussed in Chapter 9. The most efficient method should be chosen in order to minimize time and cost while meeting the accuracy requirements of a given survey project. Once a technique is chosen, the following can be set up: equipment requirements, observation schedules, and sessions designations and planning functions.

a. General equipment requirements. The type of GPS instrumentation used on a project depends on the accuracy requirements of the project, GPS survey technique, project size, and economics. Most USACE projects can be completed using a single-frequency receiver. Dual-frequency receivers are recommended as baseline lengths approach or exceed 50 km. This length may also vary depending on the amount of solar activity during the observation period. Using a dual-frequency receiver permits the user to solve for possible ionospheric and tropospheric delays which can occur as the signal travels from the satellite to the receiver antenna.

(1) Number of GPS receivers. The minimum number of receivers required to perform a differential GPS survey is two. The actual number used on a project will depend on the project size and number of available instruments/operators. Using more than two receivers will often increase productivity and allow for more efficient field observations. For some kinematic applications, two reference (set at known points) receivers and at least one rover are recommended.

(2) Personnel. Personnel requirements are also project dependent. Most GPS equipment is compact and light weight and only requires one person per station setup. However, some cases where a station is not easily accessible or requires additional power for a data link, two individuals may be required.

(3) Transportation. One vehicle is normally required for each GPS receiver used on a project. This vehicle should be equipped to handle the physical conditions that may be encountered while performing the field observations. In most cases, a two-wheel-drive vehicle should be adequate for performing all field observations. If adverse site conditions exist, a four-wheel-drive vehicle may be required. Adequate and reliable transportation is important when the observation schedule requires moving from one station to another between observation sessions.

(4) Auxiliary equipment. Adequate power should be available for all equipment (receivers, computers, lights, etc.) that will be used during the observations. Computers (386-based recommended), software, and data storage devices (floppy disks and/or cassette tapes) should be available for onsite field data reduction use. Other equipment required for conduct of a GPS survey should include tripods, tribrachs, measuring tapes, flagging, flashlights, tools, equipment cables, compass, and inclinometer. If real-time positioning is required, then a data link is also needed.

b. Observation schedules. Planning a GPS survey requires that the surveyor determine when satellites will be visible for the given survey area; therefore, the first step in determining observation schedules is to plot a satellite visibility plot for the project area. Even when the GPS becomes fully operational, full 24-hr coverage of at least four satellites may not be available in all areas.

(1) Most GPS manufacturers have software packages which at least predict satellite rise and set times. An excellent satellite plot will have the following essential information: satellite azimuths, elevations, set and rise times, and satellite PDOPs for the desired survey area. Satellite ephemeris data are generally required as input for the prediction software.

(2) To obtain broadcast ephemeris information, a GPS receiver collects data during a satellite window. The receiver antenna does not have to be located over a

known point when collecting a broadcast ephemeris. The data are then downloaded to a personal computer where they are used as input into the software prediction program. Besides ephemeris data for the software, the user is generally required to enter approximate latitude and longitude (usually scaled from a topographic map) and time offset from UTC for the survey area.

(3) From the satellite plot, the user can determine the best time to perform a successful GPS survey by taking advantage of the best combination of satellite azimuths, elevations, and PDOPs as determined by the satellite visibility plot for the desired survey area (for further information on favorable PDOP values, refer to Chapter 5). The number of sessions and/or stations per day depends on satellite visibility, travel times between stations, and the final accuracy of the survey. Often, a receiver is required to occupy a station for more than one session per day.

(4) A satellite polar plot (Figure 8-9), a satellite azimuth and elevation table (Figure 8-10), and a PDOP versus time plot (Figure 8-11) may be run prior to site reconnaissance. The output files created by the satellite prediction software are used in determining if a site is suitable for GPS surveying.

(5) Determination of session times is based mainly on the satellite visibility plan with the following factors taken into consideration: time required to permit safe travel between survey sites; time to set up and take down the equipment before and after the survey; time of survey; and possible time loss due to unforeseeable problems or complications. Station occupation during each session should be designed to minimize travel time in order to maximize the overall efficiency of the survey.

c. Session designations and planning functions. A survey session in GPS terminology refers to a single period of observation. Sessions and station designations are usually denoted by alphanumeric characters (0, 1, 2, A, B, C, etc.), determined prior to survey commencement.

(1) When only eight numeric characters are permitted for station/session designations, the following convention may be followed:

<u>12345678</u>

where

1 = type of monument with the following convention being recommended:

1 = known horizontal control monument
2 = known benchmark
3 = known 3D monument
4 = new horizontal control monument
5 = new benchmark
6 = new 3D monument
7 = unplanned occupation
8 = temporary 2D point
9 = temporary 3D point

2, 3, 4 = actual station number given to each station

5, 6, 7 = Julian day of year

8 = session number

(a) Example: Station Identifier: 40011821
Position: 12345678

(b) The numeral 4 in the number 1 position indicates the monument being established is a new monument where only horizontal position is being established.

(c) The 001 in the number 2, 3, and 4 position is the station number that has been given to the monument for this project.

(d) The 182 in the number 5, 6, and 7 position is the Julian day of the year. This is the same day as 1 July.

(e) The numeral 1 in the number 8 position identifies the session number during which observations are being made. If the receiver performed observations during the second session on the same day on the same monument, the session number should be changed to 2 for the period of the second session (then the total station identifier would be 40011822).

(2) When alpha characters are permitted for station/session designation, then a more meaningful designation can be assigned to the designation. The date of each survey session should be recorded during the survey as calendar dates and Julian days and used in the station/session designation. Some GPS software programs will require Julian dates for correct software operation. In addition to determination of station/session designations before the survey begins, the user (usually the crew chief) must:

(a) Determine the occupant of each station.

(b) Determine satellite visibility for each station.

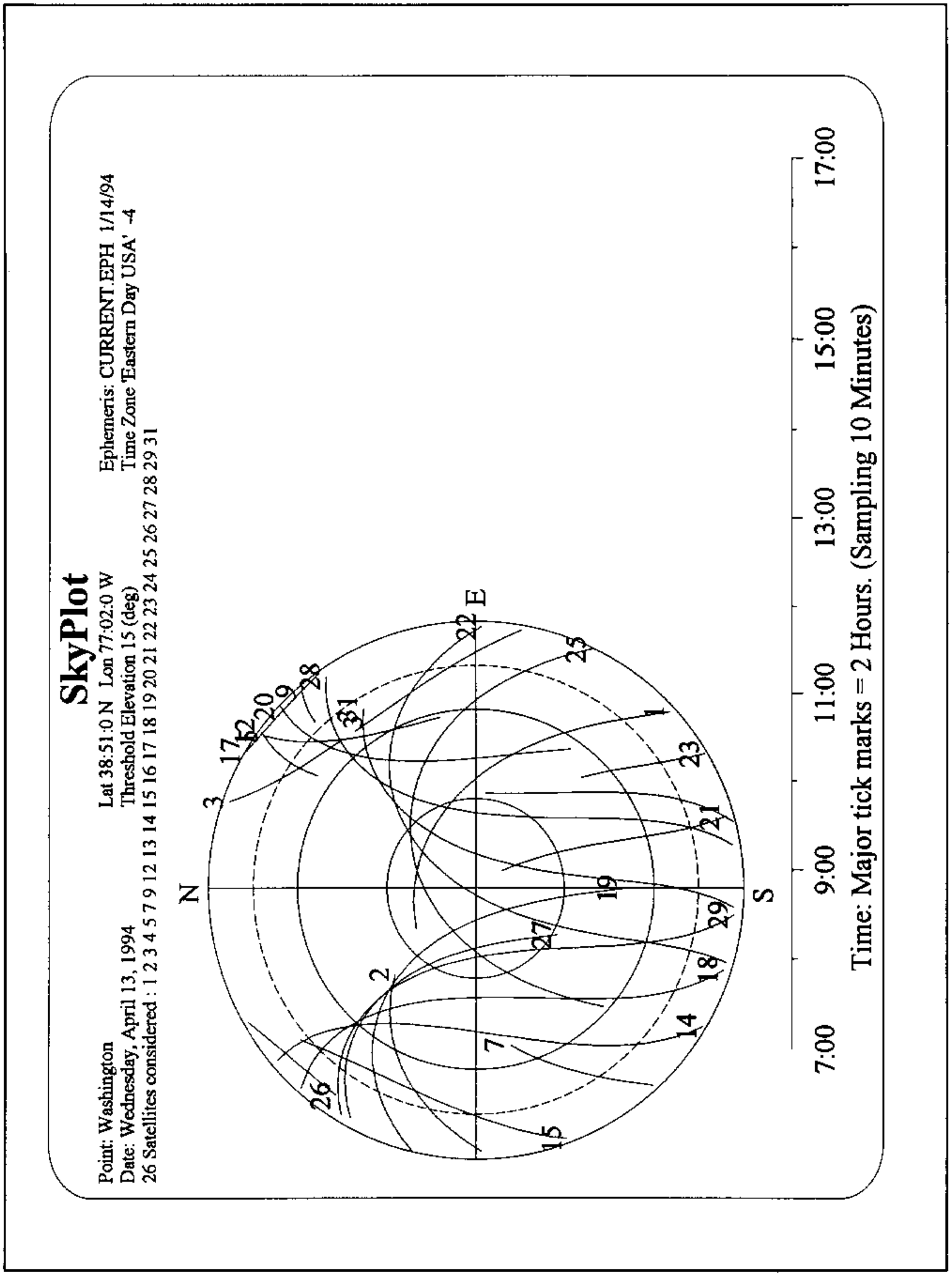


Figure 8-9. Satellite polar plot

SV Constellations

Point: Washington Lat 38:51:0 N Lon 77:02:0 W Ephemeris: CURRENT.EPH 1/14/94
Date: Wednesday, April 13, 1994 Threshold Elevation 15 (deg) Time Zone 'Eastern Day USA' -4
26 Satellites considered : 1 2 3 4 5 7 9 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 31
Sampling Rate: 10 Minutes

Constellation	T Rise	T Set	dT	PDOP Rise	PDOP Set
4 5 13 16 20 24 26	0:00	0:10	0:10	2.5	2.5
4 13 16 20 24 26	0:10	0:20	0:10	2.5	2.5
3 4 13 16 20 24 26	0:20	0:30	0:10	1.8	1.8
3 4 16 20 24 26	0:30	0:40	0:10	1.9	1.9
3 16 17 24 26	0:40	1:40	1:00	2.9	3.2
3 16 17 23 24 26	1:40	2:00	0:20	2.5	2.5
3 16 17 23 26	2:00	2:40	0:40	3.6	4.4
3 9 16 17 23 26	2:40	3:40	1:00	2.7	3.2
3 9 12 16 17 21 23 26	3:40	3:50	0:10	1.8	1.8
3 9 12 17 21 23 26	3:50	4:10	0:20	2.1	2.2
3 9 12 17 21 23 26 28	4:10	4:20	0:10	2.0	2.0
1 3 9 12 17 21 23 26 28	4:20	4:30	0:10	1.9	1.9
1 3 9 12 17 21 23 26	4:30	4:40	0:10	2.0	2.0
1 9 12 17 21 23 26	4:40	5:00	0:20	2.2	2.2
1 9 12 17 21 23	5:00	5:40	0:40	5.3	4.7
1 5 9 12 17 21 23	5:40	6:00	0:20	3.3	3.2
1 5 9 12 21 23	6:00	6:10	0:10	3.3	3.3
1 5 9 12 20 21 23	6:10	6:20	0:10	2.9	2.9
1 5 9 12 20 21 23 25	6:20	7:00	0:40	2.1	2.1
1 5 12 20 21 23 25	7:00	7:20	0:20	2.4	2.4
1 5 12 15 20 21 23 25	7:20	7:30	0:10	2.0	2.0
1 5 15 20 21 23 25	7:30	8:00	0:30	2.1	1.9
1 5 15 20 21 25	8:00	8:40	0:40	2.8	2.1
1 14 15 20 21 25	8:40	9:00	0:20	2.3	2.3
1 14 20 21 25	9:00	9:10	0:10	2.5	2.5
1 14 20 21 22 25	9:10	9:20	0:10	2.4	2.4
1 14 20 22 25	9:20	10:00	0:40	2.7	2.8
1 14 20 22 25 29	10:00	10:10	0:10	2.3	2.3
1 14 22 25 29	10:10	10:50	0:40	2.8	2.6
3 14 22 25 29	10:50	11:10	0:20	3.1	3.2
3 14 22 25 28 29	11:10	11:20	0:10	2.5	2.5
3 14 18 22 25 28 29	11:20	12:00	0:40	1.9	2.1
3 14 18 22 25 28 29 31	12:00	12:20	0:20	1.9	1.8
3 18 22 25 28 29 31	12:20	12:40	0:20	2.2	2.2
3 18 19 22 28 29 31	12:40	12:50	0:10	2.0	2.0
18 19 22 28 29 31	12:50	13:50	1:00	3.0	4.3
18 19 22 27 28 29 31	13:50	14:50	1:00	2.8	2.2
18 19 27 28 31	14:50	15:00	0:10	3.1	3.1
15 18 19 27 28 31	15:00	15:20	0:20	2.8	2.7
2 15 18 19 27 28 31	15:20	15:40	0:20	1.9	1.8
2 15 19 27 28 31	15:40	15:50	0:10	2.1	2.1
2 15 19 27 31	15:50	16:00	0:10	2.9	2.9
2 7 15 19 27 31	16:00	17:20	1:20	2.4	2.7
2 7 15 19 27	17:20	17:50	0:30	6.8	6.8
2 7 14 15 19 27	17:50	18:00	0:10	5.5	5.5
2 7 13 14 15 27	18:00	18:20	0:20	2.9	2.8
2 4 7 13 14 15 27	18:20	18:40	0:20	2.6	2.4
2 4 7 13 14 15	18:40	18:50	0:10	2.4	2.4
2 4 7 9 13 14 15	18:50	19:00	0:10	2.0	2.0
2 4 7 9 12 13 14 15	19:00	19:20	0:20	2.1	2.3
2 4 7 9 12 13 14	19:20	19:30	0:10	2.8	2.8
2 4 7 9 12 13 14 24	19:30	20:20	0:50	2.2	2.2
2 4 5 7 9 12 13 14 24	20:20	20:50	0:30	1.9	1.7
2 4 5 7 12 13 24	20:50	21:00	0:10	2.5	2.5
4 5 7 12 13 24	21:00	21:30	0:30	3.4	3.7
4 5 7 12 13 18 24	21:30	21:40	0:10	2.9	2.9
4 5 7 12 13 16 18 24	21:40	21:50	0:10	2.7	2.7
4 5 7 12 13 16 18 20 24	21:50	22:00	0:10	1.7	1.7
4 5 7 13 16 18 20 24	22:00	22:30	0:30	1.9	2.0
4 5 13 16 18 20 24	22:30	23:20	0:50	2.4	2.6
4 5 13 16 20 24	23:20	23:50	0:30	3.0	2.7
4 5 13 16 20 24 26	23:50	24:00	0:10	2.5	2.5

Figure 8-10. Satellite azimuth and elevation table

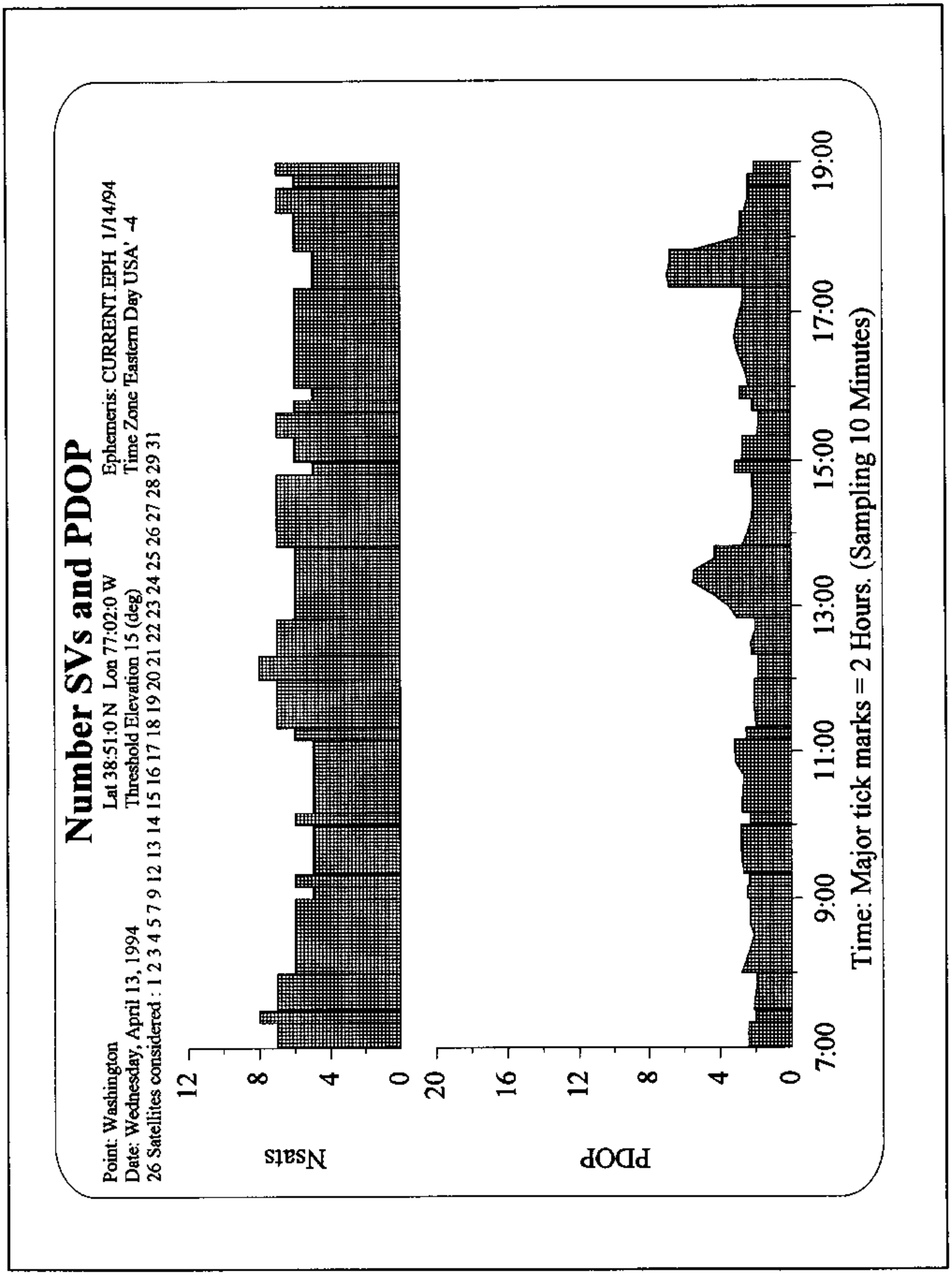


Figure 8-11. PDOP versus time plot

(c) Require site reconnaissance data for stations to be occupied. Remember the same person who performed the initial site reconnaissance may not be the individual performing the survey; therefore, prior determined site reconnaissance data may require clarification before survey commencement.

(d) Develop a project sketch.

(e) Issue explicit instructions on when each session is to begin and end.

(f) Require a station data logging sheet completed for each station. Figures 8-12 and 8-13 are examples of various station logs used in USACE, along with blank forms which may be used as worksheets. Standard bound field survey books may be used in lieu of separate log/work sheets.

U.S. ARMY CORPS OF ENGINEERS
GPS DATA LOGGING SHEET

PROJECT NAME COYOTE DAM LOCALITY UKIAH, CA
OBSERVER LARRY LAMB AGENCY/FIRM COE, SACRAMENTO DIST.
RECEIVER TRIMBLE 4000 SL S/N 2820A00223
ANTENNA TRIMBLE MICRO SL S/N 2816A00224
DATA RECORDING UNIT RECEIVER S/N 2820A00223
TRIBRACH WILD GDF22 S/N N/A LAST CALIBRATED: 4/24/89

	SESSION 1	SESSION 2	SESSION 3
STATION NAME	PIER 2	PIER 2	PIER 2
STATION NUMBER	2002	2002	2002
DAY OF YEAR	115	115	115
DATE MM DD YY	04/25/89	04/25/89	04/25/89
UTC TIME OF OBSERVATION	START 04:56 STOP 05:55	START 06:10 STOP 07:38	START 07:55 STOP 09:20

ANTENNA HEIGHT MEASUREMENTS

	SESSION 1	SESSION 2	SESSION 3
SLOPE @ BEGINNING	0.120 0.120 0.119 4 13/16 IN = 0.121 M MN = 0.120 M	0.116 0.116 0.116 4 9/16 IN = 0.116 M MN = 0.116 M	0.123 0.124 0.124 4 14/16 IN = 0.124 M MN = 0.1238 M
SLOPE @ END	4 11/16 4 13/16 4 11/16 0.120 M = 4 13/16 IN MN = 0.120 M	4 9/16 4 9/16 4 9/16 0.116 M = 4 9/16 IN MN = 0.116 M	4 13/16 4 14/16 4 14/16 0.123 M = 4 13/16 IN MN = 0.1230 M
MN ADJ TO VERT	0.120 M	0.116 M	0.1234 M

	PROGRAMMED REFPOS	FIELD POSITION	PROGRAMMED REFPOS	FIELD POSITION	PROGRAMMED REFPOS	FIELD POSITION
LAT	39-12-30	39-12-22.64	39-12-30	39-12-22.48	39-12-30	39-12-22.81
LONG	123-10-30	123-10-33.42	123-10-30	123-10-33.20	123-10-30	123-10-33.62
HT	244.0	210.6	244.0	199.8	244.0	222.8
PDOP	3.6		4.8		4.0	
SVS TO TRACK	02, 03, 06, 09, 11, 12, 13, 14		02, 03, 06, 09, 11, 12, 13, 14		03, 06, 09, 11, 12, 13, 14, 16	

LOCAL TIME:	SCHEDULED	ACTUAL	SCHEDULED	ACTUAL	SCHEDULED	ACTUAL
START	21:55	21:56	23:38	23:10	01:20	00:55
STOP	22:55	22:55	00:38	00:38	02:20	02:20

PAGE 1

Figure 8-12. Sample GPS data logging sheet (Continued)

U.S. ARMY CORPS OF ENGINEERS
GPS DATA LOGGING SHEET

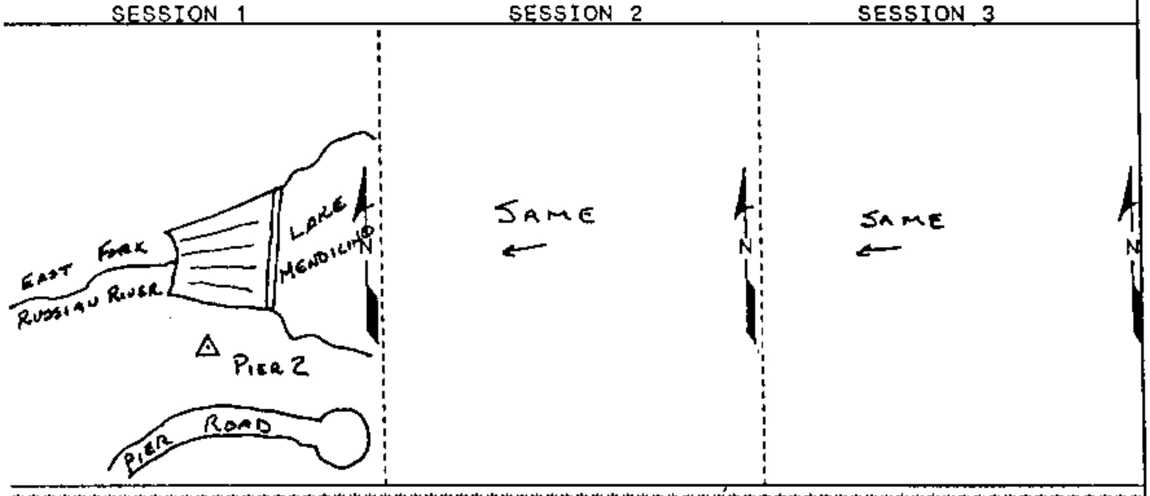
****

	SESSION 1	SESSION 2	SESSION 3
ANT CABLE LENGTH	100 ft	100 ft	35 ft
POWER SUPPLY	12v DC	12v DC	12v DC
WEATHER CONDITIONS	Clear, Cool 45°	Clear, Cool 40°	Clear, Breezy 40°
MONUMENT TYPE	'C' (Set in Pier)	← Same	Same
EXACT STAMPING	Pier 2 1953	← "	"
AGENCY CAST IN DISK	COE	← "	"

****

SKETCH OF SITE

SESSION 1	SESSION 2	SESSION 3



****

Describe any abnormalities and/or problems encountered during the survey, include session number, time of occurance and duration.

The Antenna Was Mounted Directly over Pier 2 With No Tripod Used.

Antenna Height Was Measured Vertically From Ground Plane To Brass Disk.

****

PAGE 2

Figure 8-12. (Concluded)

U.S. ARMY CORPS OF ENGINEERS
GPS DATA LOGGING SHEET

****
PROJECT NAME ______________________ LOCALITY ____________________
OBSERVER ______________________ AGENCY/FIRM ____________________
RECEIVER __________________________ S/N ____________________
ANTENNA ___________________________ S/N ____________________
DATA RECORDING UNIT ______________________ S/N ____________________
TRIBRACH ______________ S/N __________ LAST CALIBRATED: __________
****

	SESSION 1		SESSION 2		SESSION 3	
STATION NAME	________		________		________	
STATION NUMBER	________		________		________	
DAY OF YEAR	________		________		________	
DATE MM DD YY	________		________		________	
UTC TIME OF OBSERVATION	START	STOP	START	STOP	START	STOP

****

ANTENNA HEIGHT MEASUREMENTS

	SESSION 1	SESSION 2	SESSION 3
SLOPE @ BEGINNING	____ ____ ____ ____ IN =____ M MN = ______ M	____ ____ ____ ____ IN =____ M MN = ______ M	____ ____ ____ ____ IN =____ M MN = ______ M
SLOPE @ END	____ ____ ____ ____ M =____ IN MN = ______ M	____ ____ ____ ____ M =____ IN MN = ______ M	____ ____ ____ ____ M =____ IN MN = ______ M
MN ADJ TO VERT	________ M	________ M	________ M

****

	PROGRAMMED REFPOS	FIELD POSITION	PROGRAMMED REFPOS	FIELD POSITION	PROGRAMMED REFPOS	FIELD POSITION
LAT	______	______	______	______	______	______
LONG	______	______	______	______	______	______
HT	______	______	______	______	______	______
PDOP	______		______		______	
SVS TO TRACK	______ ______		______ ______		______ ______	
LOCAL TIME:	SCHEDULED	ACTUAL	SCHEDULED	ACTUAL	SCHEDULED	ACTUAL
START	______	______	______	______	______	______
STOP	______	______	______	______	______	______

****

PAGE 1

Figure 8-13. Worksheet 8-3, GPS data logging sheet (Continued)

U.S. ARMY CORPS OF ENGINEERS
GPS DATA LOGGING SHEET

****

	SESSION 1	SESSION 2	SESSION 3
ANT CABLE LENGTH	________	________	________
POWER SUPPLY	____________	____________	____________
WEATHER CONDITIONS	____________	____________	____________
MONUMENT TYPE	____________	____________	____________
EXACT STAMPING	____________	____________	____________
AGENCY CAST IN DISK	____________	____________	____________

****

SKETCH OF SITE

SESSION 1	SESSION 2	SESSION 3
N	N	N

****

Describe any abnormalities and/or problems encountered during the survey, include session number, time of occurance and duration.

****

PAGE 2

Figure 8-13. (Concluded)

Chapter 9
Conducting GPS Field Surveys

Section I
Introduction

9-1. General

This chapter presents guidance to field personnel performing GPS surveys for all types of USACE projects. The primary emphasis in this chapter is on static and kinematic carrier phase differential GPS measurements which is covered in Section IV. Absolute positioning is covered in Section II. Section III covers differential code phase GPS positioning techniques.

9-2. General GPS Field Survey Procedures

The following are some general GPS field survey procedures that should be performed at each station, observation, and/or session on a GPS survey.

a. Receiver setup. GPS receivers shall be set up in accordance with manufacturer's specifications prior to beginning any observations. To eliminate any possibility of missing the beginning of the observation session, all equipment should be set up with power supplied to the receivers at least 10 min prior to the beginning of the observation session. Most receivers will lock-on to satellites within 1-2 min of powering up.

b. Antenna setup. All tribrachs used on a project should be calibrated and adjusted prior to beginning each project. Dual use of both optical plummets and standard plumb bobs is strongly recommended since centering errors represent a major error source in all survey work, not just GPS surveying.

c. Height of instrument measurements. Height of instrument (HI) refers to the correct measurement of the distance of the GPS antenna above the reference monument over which it has been placed. HI measurements will be made both before and after each observation session. The HI will be made from the monument to a standard reference point on the antenna. (See Figure 9-1.) These standard reference points for each antenna will be established prior to the beginning of the observations so all observers will be measuring to the same point. All HI measurements will be made both in meters and feet for redundancy and blunder detection. HI measurements shall be determined to the nearest millimeter in metric units and to the nearest 0.01 ft (or 1/16 in.). It should be noted whether the HI is vertical or diagonal.

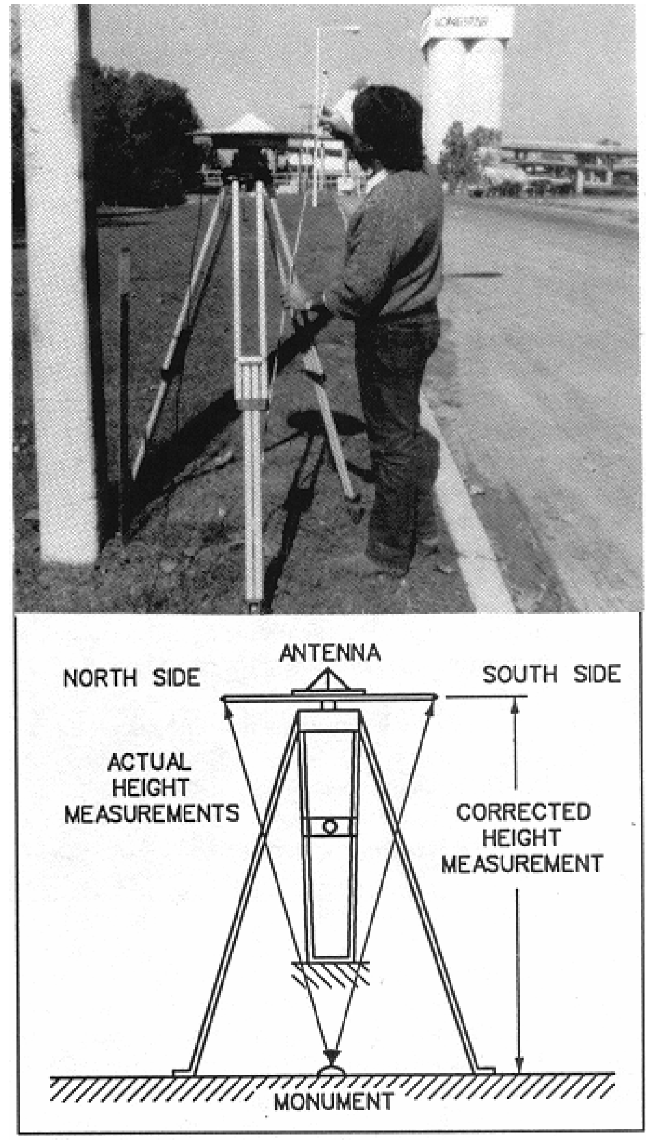


Figure 9-1. Height of instrument measurement setup

d. Field GPS observation recording procedures. Field recording books, log sheets, or log forms will be completed for each station and/or session. Any acceptable recording media may be used. For archiving purposes, standard bound field survey books are preferred; however, USACE Commands may require specific recording sheets/ forms to be used in lieu of a survey book. The amount of record-keeping detail will be project-dependent; low-order topographic mapping points need not have as much descriptive information as would permanently marked primary control points. The following typical data may be included on these field log records:

(1) Project, construction contract, observer(s) name(s), and/or contractor firm and contract number.

(2) Station designation.

(3) Station file number.

(4) Date, weather conditions, etc.

(5) Time start/stop session (local and UTC).

(6) Receiver, antenna, data recording unit, and tribrach make, model, and serial numbers.

(7) Antenna height: vertical or diagonal measures in inches (or feet) and meters (or centimeters).

(8) Space vehicle designations (satellite number).

(9) Sketch of station location.

(10) Approximate geodetic location and elevation.

(11) Problems encountered.

USACE Commands may require that additional data be recorded. These will be contained in individual project instructions or contract delivery order scopes. Samples of typical GPS recording forms are shown later in this chapter.

e. Field processing and verification. It is strongly recommended that GPS data processing and verification be performed in the field where applicable. This is to identify any problems that may exist which can be corrected before returning from the field. Processing and verification is covered in Chapters 10 and 11.

Section II
Absolute GPS Positioning Techniques

9-3. General

The accuracy obtained by GPS point positioning is dependent on the user's authorization. The SPS user can provide an accuracy of 80-100 m. SPS data are most often expressed in real time; however, the data can be post-processed if station occupation was over a period of time. The post-processing produces a best-fit point position. Although this will provide a better internal approximation, the effects of S/A when activated still degrade positional accuracy up to 80-100 m. The PPS user requires a decryption device within the receiver to decode the effects of S/A. The PPS provides an accuracy between 10 and 16 m when a single-frequency receiver is used for observation. Dual-frequency receivers using the precise ephemeris may produce an absolute positional accuracy on the order of 1 m or better. These positions are based on the absolute WGS 84 ellipsoid. The PPS that uses the precise ephemeris requires the data to be post-processed. At present, a commercial or military receiver capable of meter-level GPS point positioning without post-processing is not available.

9-4. Absolute (Point Positioning) Techniques

There are two techniques used for point positioning in the absolute mode. They are long-term averaging of positions and differencing between signals.

a. In long-term averaging, a receiver is set up to store positions over a period of observation time. The length of observation time varies based upon the accuracy required. The longer the period of data collection, the better average position. These observation times can range between 1 and 24 hr. This technique can also be used in real-time (i.e., the receiver averages the positions as they are calculated). For example, the precise lightweight GPS receiver (PLGR) GPS receiver uses this technique in calculating a position at a point.

b. The process of differencing between signals can only be performed in a post-processed mode. Currently, the Defense Mapping Agency has produced software that can perform this operation.

Section III
Differential Code Phase GPS Positioning Techniques

9-5. General

Differential (or relative) GPS surveying is the determination of one location with respect to another location. When using this technique with the C/A- or P-code it is called relative code phase positioning or surveying. Relative code phase positioning has limited application to detailed engineering surveying and topographic site plan mapping applications. Exceptions include general reconnaissance surveys, hydrographic survey vessel or dredge positioning (see EM 1110-2-1003 for further information on these surveys), and some operational military or geodetic survey support functions. Additional applications for relative code phase positioning have been on the increase as positional accuracies have become better.

9-6. Relative Code Phase Positioning

The code phase tracking differential system is currently a functional GPS survey system for positioning hydrographic survey vessels and dredges. It also has application for topographic, small-scale mapping surveys and input to a GIS database. The basic concept is shown in Figure 9-2. Although greater positional accuracies can be obtained with use of the P-code, DoD's implementation of A/S will limit its use. A real-time dynamic DGPS positioning system includes a reference station, communication link, and user (remote) equipment. If results are not required in real-time, the communication link can be eliminated and the positional information is post-processed.

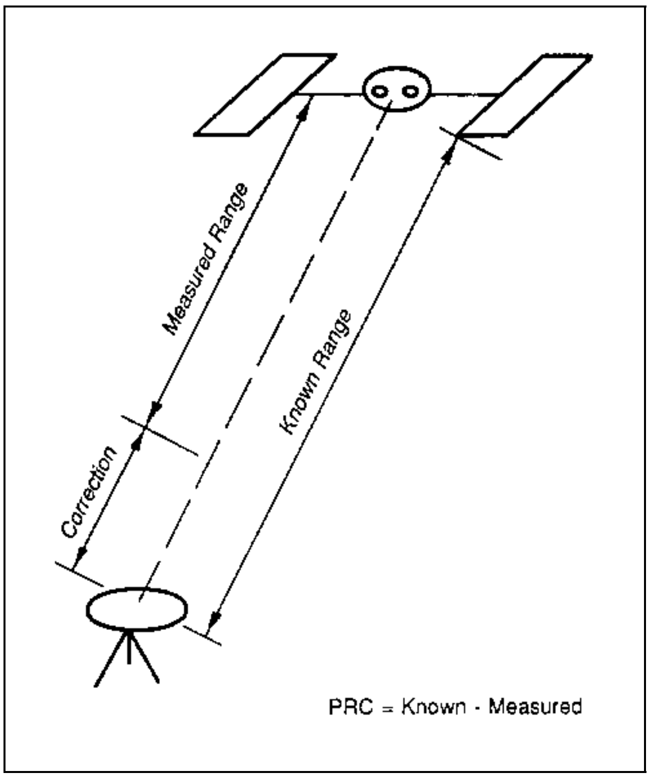


Figure 9-2. Code phase DGPS concept

a. Accuracy of relative code surveys. Relative code phase surveys can obtain accuracies of 0.5 to 10 m. These accuracies will meet Class 1 hydrographic survey standards as stated in EM 1110-2-1003. This type of survey could also be used for small-scale mapping or used as input to a GIS database.

b. Reference station. The reference station is placed on a known survey monument in an area having an unobstructed view of the sky of at least four satellites, 10 deg above the horizon. It consists of a GPS receiver, GPS antenna, processor, and a communication link (if real-time results are desired). The reference station measures the timing and ranging information broadcast by the satellites and computes and formats range corrections for broadcast to the user equipment. Using the technology of differential pseudo-ranging, the position of a survey vessel is found relative to the reference station. The pseudo-ranges are collected by the GPS receiver and transferred to the processor where PRCs are computed and formatted for data transmission. Many manufacturers have incorporated the processor within the GPS receiver, eliminating the need for an external processing device. The recommended data format is that proposed by the RTCM Special Committee (SC) 104 v. 2.0. The processor should be capable of computing and formatting PRCs every 1-3 sec. A longer time span could affect the user's positional solution due to effects of S/A.

c. Communication link. The communication link is used as a transfer media for differential corrections. The main requirement of the communication link is that transmission be at a minimum rate of 300 bits per second. The type of communication system is dependent on the user's requirements.

(1) Frequency authorization. All communication links necessitate a reserved frequency for operation to avoid interference with other activities in the area. No transmission can occur over a frequency until the frequency has been officially authorized for use in transmitting digital data. This applies to all government agencies. Allocating a frequency is handled by the FOA's Frequency Manager responsible for the area of application, the vendor supplying the equipment, and the user.

(2) Ultra High Frequency (UHF) and Very High Frequency (VHF). Communication links operating at UHF and VHF are viable systems for the broadcast of DGPS corrections. UHF and VHF can extend out some 20 to 50 km, depending on local conditions. The disadvantages of UHF and VHF links are their limited range to line of sight and the effects of signal shadowing (i.e. islands, structures, and buildings), multipath and licensing issues.

(3) Satellite communications. There are several companies that sell satellite communication systems which can be used for the transmission of PRCs. These systems can be efficient for wide areas, but are usually higher in price.

(4) License-free radio-modems. Several companies have developed low wattage (1 watt or less) radio-modems to transmit digital data. These radio-modems require no license and can be used to transmit DGPS corrections in a localized area (within 5-8 km or less depending on line of sight). The disadvantages are the short range and line-of-sight limitations.

d. User (remote station) equipment. The remote receiver should be a multichannel single frequency C/A-code GPS receiver. The receiver must be able to store the raw data to be post-processed. During post-processing, these PRCs are generated with the GPS data from the reference station and then applied to the remote station data to obtain a corrected position. If the results are desired in real time, the receiver must be able to accept the PRCs from the reference station (via data link) in the RTCM SC 104 v. 2.0 format and apply those corrections to the measured pseudo-range. The corrected position can then be input into a data collector, hydro package, or GIS database.

e. USCG DGPS Navigation Service. The USCG DGPS Navigation Service was developed to provide a nationwide (coastal regions, Great Lakes regions, and some inland waterways), all-weather, real-time, radio navigation service in support of commercial and recreational maritime interests. A 50+ station network will be operational by FY96. Its accuracy was originally designed to fulfill an 8- to 20-m maritime navigation accuracy. However, a reconfigured version of the USCG system will now yield 1.5-m 2DRMS at distances upward of 150 km from the reference beacon. The system operates on the USCG marine radio beacon frequencies (285-325 kHz). Each radio beacon has an effective range of 150 to 250 km at a 99.9 percent signal availability level. It is fully expected that the USCG system, once completed will be the primary marine navigation device used by commercial and recreational vessels requiring meter-level accuracy.

(a) Corps-wide implementation and use of the USCG system will eliminate need for maintaining existing USACE-operated microwave positioning systems. It will also significantly reduce or eliminate USACE requirements to develop independent UHF/VHF DGPS networks for meter-level vessel navigation and positioning.

(b) The USCG system has potential for supporting other nonmarine activities such as master planning, engineering, mapping, operations, and GIS development activities where meter-level accuracy is sufficient.

Section IV
Differential Carrier Phase GPS Horizontal Positioning Techniques

9-7. General

Differential (or relative) GPS carrier phase surveying is used to obtain the highest precision from GPS and has direct application to most USACE military construction and civil works topographic and engineering survey activities.

a. Differential survey techniques. There are basically six different GPS differential surveying techniques (paragraph 6-4) in use today:

(1) Static.

(2) Pseudo-kinematic.

(3) Stop and go kinematic.

(4) Kinematic.

(5) Rapid static.

(6) On-the-fly (OTF)/Real-time kinematic (RTK).

Procedures for performing each of these methods are described below. These procedures are guidelines for conducting a field survey. Manufacturers' procedures should be followed, when appropriate, for conducting a GPS field survey. Project horizontal control densification can be performed using any one of these methods. Procedurally, all six methods are similar in that each measures a 3D baseline vector between a receiver at one point (usually of known local project coordinates) and a second receiver at another point, resulting in a vector difference between the two points occupied. The major distinction between static and kinematic baseline measurements involves the method by which the carrier wave integer cycle ambiguities are resolved; otherwise they are functionally the same process.

b. Ambiguity resolution. Cycle ambiguity is the unknown number of whole carrier wavelengths between the satellite and receiver. It is also referred to as "Integer

Ambiguity." Figure 9-3 shows an example of an integer ambiguity measurement. Successful ambiguity resolution is required for successful baseline formulations. Generally, in static surveying, instrumental error and ambiguity resolution can be achieved through long-term averaging and simple geometrical principles, resulting in solutions to a linear equation that produces a resultant position. But ambiguity resolution can also be achieved through a combination of the pseudo-range and carrier beat measurements, made possible by a knowledge of the PRN modulation code.

c. Post-observation data reduction. Currently, all carrier phase relative surveying techniques, except OTF and RTK, require post-processing of the observed data to determine the relative baseline vector differences. OTF and RTK can be performed in real-time or in the post-processed mode. Post-processing of observed satellite data involves the differencing of signal phase measurements recorded by the receiver. The differencing process reduces biases in the receiver and satellite oscillators and is performed in a computer. When contemplating the purchase of a receiver, the user should keep in mind the computer requirements necessary to post-process the GPS data. Most manufacturers require, as a minimum, a 386-based IBM-compatible personal computer (PC) with a math co-processor. It is also strongly recommended that all baseline reductions be performed in the field, if possible, in order to allow an onsite assessment of the survey adequacy.

9-8. Static GPS Survey Techniques

Static GPS surveying is perhaps the most common method of densifying project network control. Two GPS receivers are used to measure a GPS baseline distance. The line between a pair of GPS receivers from which simultaneous GPS data have been collected and processed is a vector referred to as a baseline. The station coordinate differences are calculated in terms of a 3D, earth-centered coordinate system that utilizes X-, Y-, and Z-values based on the WGS 84 geocentric ellipsoid model. These coordinate differences are then subsequently shifted to fit the local project coordinate system.

a. General. GPS receiver pairs are set up over stations of either known or unknown location. Typically one of the receivers is positioned over a point whose coordinates are known (or have been carried forward as on a traverse), and the second is positioned over another point

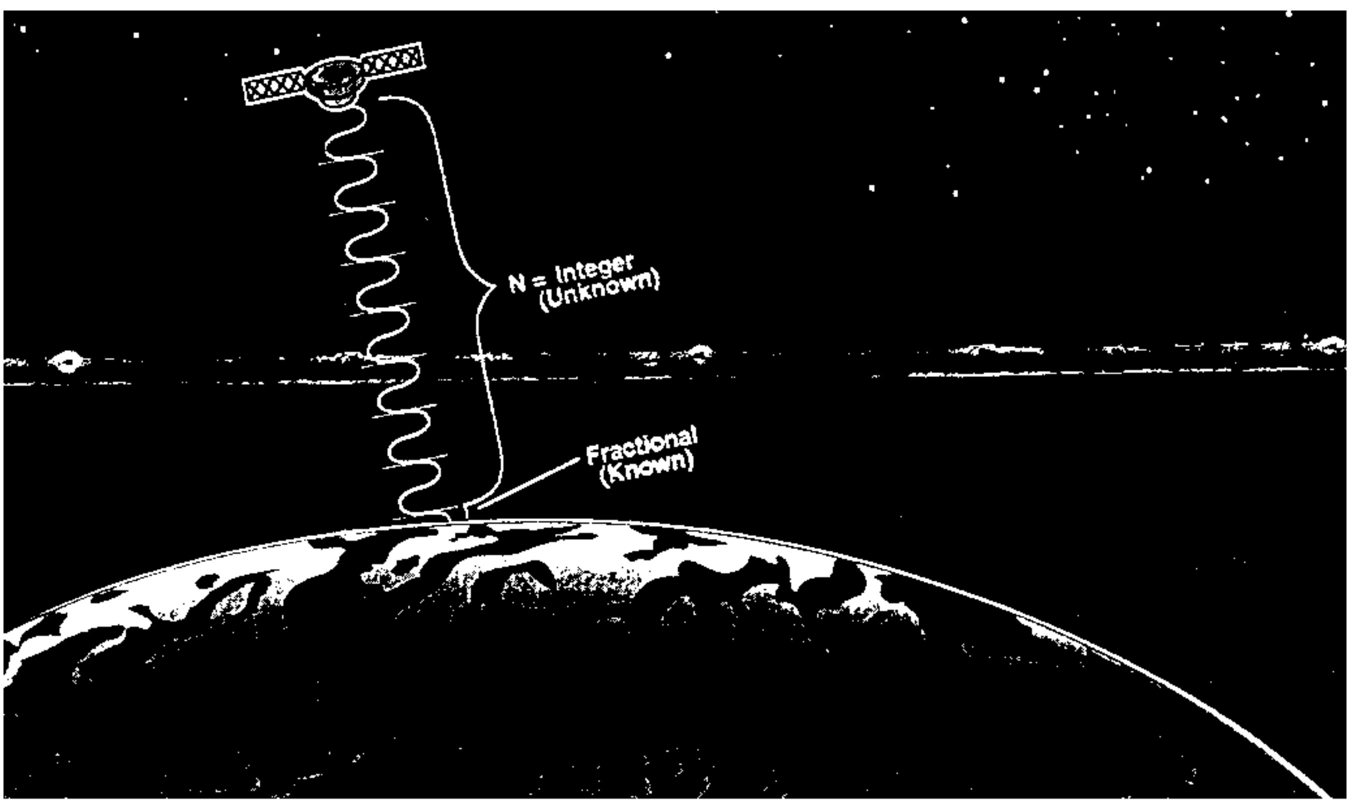


Figure 9-3. Integer Ambiguity

whose coordinates are unknown, but are desired. Both GPS receivers must receive signals from the same four (or more) satellites for a period of time that can range from a few minutes to several hours, depending on the conditions of observation and precision required.

b. Static baseline occupation time. Station occupation time is dependent on baseline length, number of satellites observed, and the GPS equipment used. In general, 30 min to 2 hr is a good approximation for baseline occupation time for shorter baselines of 1-30 km. A rough guideline developed by Trimble, Inc., for estimating occupation time is shown in Figure 9-4. Note that this guideline exceeds the recommended minimum observing times prescribed in Table 8-1.

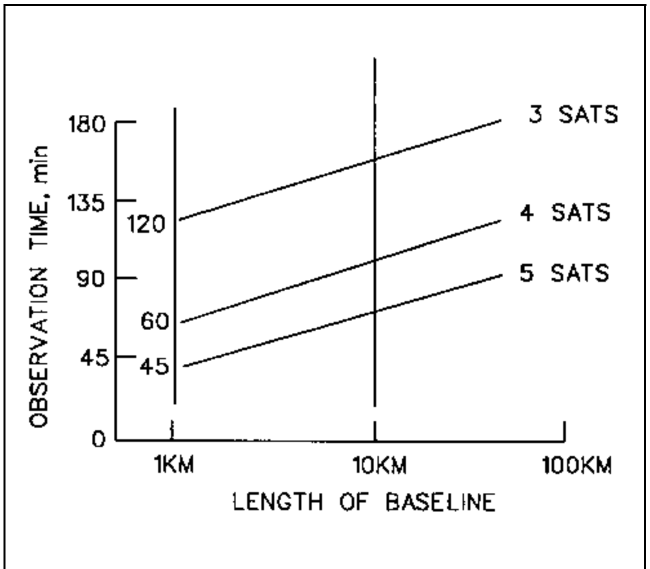


Figure 9-4. Station occupation time versus baseline distance

(1) Since there is no definitive guidance for determining the required baseline occupation time, the results from the baseline reduction (and subsequent adjustments) will govern the adequacy of the observation irrespective of the actual observation time. The most prudent policy is to exceed the minimum estimated times, especially for lines where reoccupation would be difficult or field data assessment capabilities are limited.

(2) For baselines greater than 50 km in length, the ionosphere may have an adverse effect on the solution. Adverse ionosphere effects for baselines of this length can be reduced by using a dual-frequency GPS receiver, as opposed to a single frequency as is normally used.

c. Satellite visibility requirements. The stations that are selected for survey must have an unobstructed view of the sky for at least 15 deg or greater above the horizon during the "observation window." An observation window is the period of time when observable satellites are in the sky and the survey can be successfully conducted.

d. Common satellite observations. It is critical for a static survey baseline reduction/solution that the receivers simultaneously observe the same satellites during the same time interval. For instance, if receiver No. 1 observes a satellite set during the time interval 1,000 to 1,200 and another receiver, receiver No. 2, observes that same satellite set during the time interval 1,100 to 1,300, only the period of common observation, 1,100 to 1,200, can be processed to formulate a correct vector difference between these receivers.

e. Data post-processing. After the observation session has been completed, the received GPS signals from both receivers are then processed (i.e., "post-processed") in a computer to calculate the 3D baseline vector components between the two observed points. From these vector distances, local or geodetic coordinates may be computed and/or adjusted.

f. Survey configuration. Static baselines may be extended from existing control using any of the control densification methods described in Chapter 8. These include networking, traverse, spur techniques, or combinations thereof. Specific requirements are normally contained in project instructions (or scope of work) provided by the District office.

g. Receiver operation and data reduction. Specific receiver operation and baseline data post-processing requirements are very manufacturer-dependent. The user is strongly advised to consult and study manufacturer's operations manuals thoroughly along with the baseline data reduction examples shown in this manual.

h. Accuracy of static surveys. Accuracy of GPS static surveys will usually exceed 1 ppm. Currently of all GPS processing methods, static is the most accurate and can be used for any order survey.

9-9. Stop-and-Go Kinematic GPS Survey Techniques

Stop-and-go surveying is similar to static surveying in that each method requires at least two receivers simultaneously recording observations. A major difference between static and stop-and-go surveying is the amount of time required for a receiver to stay fixed over a point of unknown position. In stop-and-go surveying, the first receiver--the home or reference receiver--remains fixed on a known control point. The second receiver--the “rover” receiver--collects observations statically on a point of unknown position for a period of time (usually a few minutes), and then moves to subsequent unknown points to collect signals for a short period of time. During the survey, at least four common satellites (preferably five) need to be continuously tracked by both receivers. Once all required points have been occupied by the rover receiver, the observations are then post-processed by a computer to calculate baseline vector/coordinate differences between the known control point and points occupied by the rover receiver during the survey session. The main advantage of this form of GPS surveying over static surveying is the reduced occupation time required over the unknown points. Because stop-and-go surveying requires less occupation time over unknown points, time and cost for the conduct of a survey are significantly reduced. Achievable accuracies typically equal or exceed Third-Order, which is adequate for most USACE projects.

a. Survey procedure. A typical stop-and-go survey scheme is illustrated in Figure 9-5. Stop-and-go GPS surveying is performed similarly to a conventional EDM traverse or electronic total station radial survey. The system is initially calibrated by performing either an antenna swap (see *d* below) with one known point and one unknown point or by performing a static measurement over a known baseline. This calibration process is performed to resolve initial cycle ambiguities. This known baseline may be part of the existing network or can be established using static GPS survey procedures described above. The remote roving receiver then traverses between unknown points as if performing a radial topographic survey. Typically, the points are double-connected, or double-run, as in a level line. Optionally, two fixed receivers may be used to provide redundancy on the remote points. With only 1-1/2 min at a point, X-Y-Z coordinate production is high and limited only by satellite observing windows, travel time between points, and overhead obstructions.

b. Satellite lock. During a stop-and-go kinematic survey, the rover station must maintain lock on at least four satellites during the period of survey (the reference station must be observing at least the same four satellites). Loss of lock occurs when the receiver is unable to continuously record satellite signals or the transmitted satellite

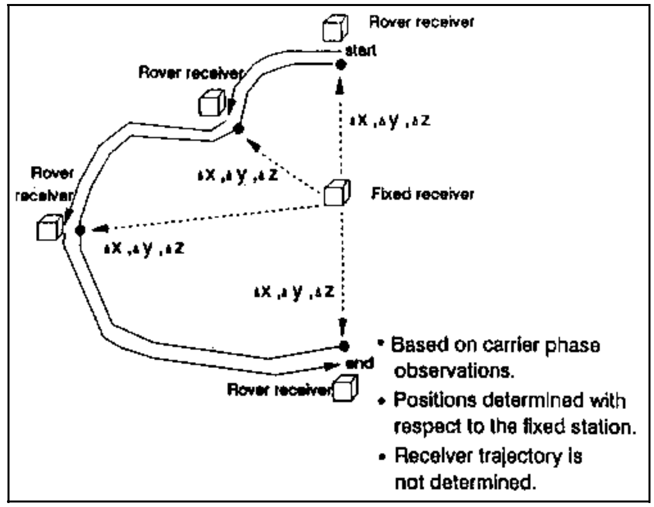


Figure 9-5. Typical stop-and-go survey scheme

signal is disrupted and the receiver is not able to record it. If satellite lock is lost, the roving receiver must reobserve the last control station surveyed before loss of lock. The receiver operator must monitor the GPS receiver when performing the stop-and-go survey to ensure loss of lock does not occur. Some manufacturers have now incorporated an alarm into their receiver that warns the user when loss of lock occurs, thus making the operator’s job of monitoring the receiver easier.

c. Site constraints. Survey site selection and route between rover stations to be observed are critical. All sites must have a clear view of satellites having a vertical angle of 15 deg or greater. The routes between rover occupation stations must be clear of obstructions so that the satellite signal is not interrupted. Each unknown station to be occupied should be occupied for a minimum of at least 1-1/2 min. Stations should be occupied two or three times to provide redundancy between observations.

d. Antenna swap calibration procedure. Although the antenna swap procedure can be used to initialize a survey prior to a stop-and-go survey, an antenna swap can also be used to determine a precise baseline and azimuth between two points. The procedure requires that both stations occupied and the path between both stations maintain an unobstructed view of the horizon. A minimum of four satellites and maintainable lock are required to perform an antenna swap; however, more than four satellites are preferred. To perform an antenna swap, one receiver/antenna is placed over a point of known control and the second, a distance of 10 to 100 m away from the other receiver. Referring to Figure 9-6, the receivers at each station collect data for approximately 2 to 4 min. The receivers/antennae sets then swap locations; the receiver/antenna at the known station is moved to the unknown site while the other receiver/antenna at the unknown site is moved to the known site. Satellite data are again collected for 2 to 4 min. The receivers are then swapped back to their original locations. This completes one antenna swap calibration. If satellite lock is lost during the procedure, the procedure must be repeated.

e. Accuracy of stop-and-go surveys. Accuracy of stop-and-go baseline measurements will usually well exceed 1 part in 5,000; thus, Third-Order classification project/mapping horizontal control can be effectively, efficiently, and accurately established using this technique. For many USACE projects, this order of horizontal accuracy will be more than adequate; however, field procedures should be designed to provide adequate redundancy for what are basically “open-ended” or “spur” points. Good satellite geometry and minimum multipath are also essential in performing acceptable stop-and-go surveys.

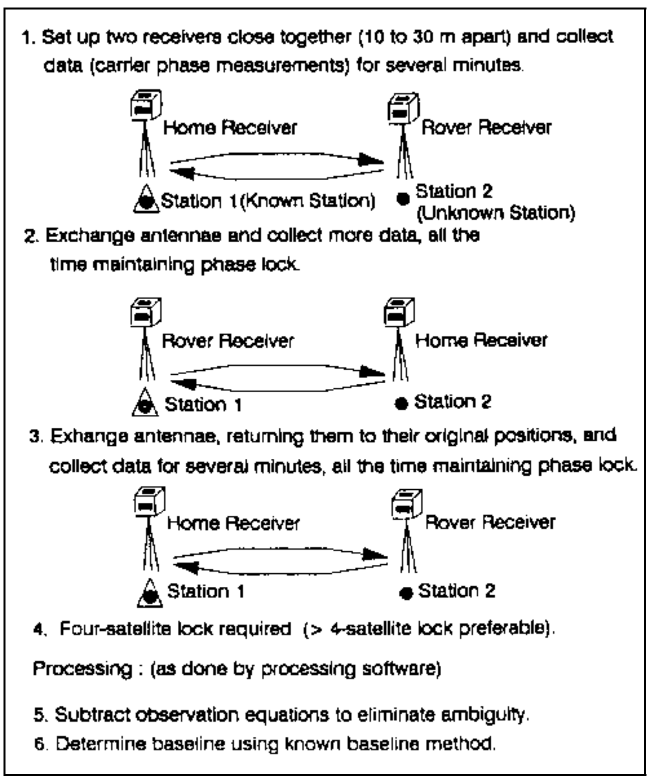


Figure 9-6. Stop-and-go ambiguity resolution (antenna swap method)

9-10. Kinematic GPS Survey Techniques

Kinematic surveying using differential carrier phase tracking is similar to the two previous types of differential carrier phase GPS surveying because it also requires two receivers recording observations simultaneously. Kinematic surveying is often referred to as dynamic surveying. As in stop-and-go surveying, the reference receiver remains fixed on a known control point while the roving receiver collects data on a constantly moving platform (vehicle, vessel, aircraft, manpack, etc.), as illustrated in Figure 9-7. Unlike stop-and-go surveying, kinematic surveying techniques do not require the rover receiver to remain motionless over the unknown point. The observation data are later post-processed with a computer to calculate relative vector/coordinate differences to the roving receiver.

a. Survey procedure. A kinematic survey requires two single frequency (L1) receivers. One receiver is set over a known point (reference station) and the other is used as a rover (i.e., moved from point to point or along a path). Before the rover receiver can rove, a period of static initialization or antenna swap (see paragraph 9-9*d*) must be performed. This period of static initialization is dependent on the number of satellites visible. Once this is done, the rover receiver can move from point to point as long as satellite lock is maintained on at least four common (with the reference station) satellites. If loss of satellite lock occurs, a new period of static initialization must take place. It is important to follow manufacturers' specifications when performing a kinematic survey.

b. Kinematic data processing techniques. In general, kinematic data processing techniques are similar to those used in static surveying (Chapter 10). When processing kinematic GPS data, the user must ensure that satellite lock was maintained on four or more satellites and that cycle slips are adequately resolved in the data recorded.

c. Accuracy of kinematic surveys. Differential (carrier phase) kinematic survey errors are correlated between observations received at the reference and rover receivers, as in differential static surveys. Experimental test results indicate kinematic surveys can produce results in centimeters. Test results from an experimental full kinematic GPS survey conducted by U.S. Army Engineer Topographic Laboratory (now TEC) personnel at White Sands Missile Range, Holloman Air Force Base, New Mexico, verified (under ideal test conditions) that kinematic GPS

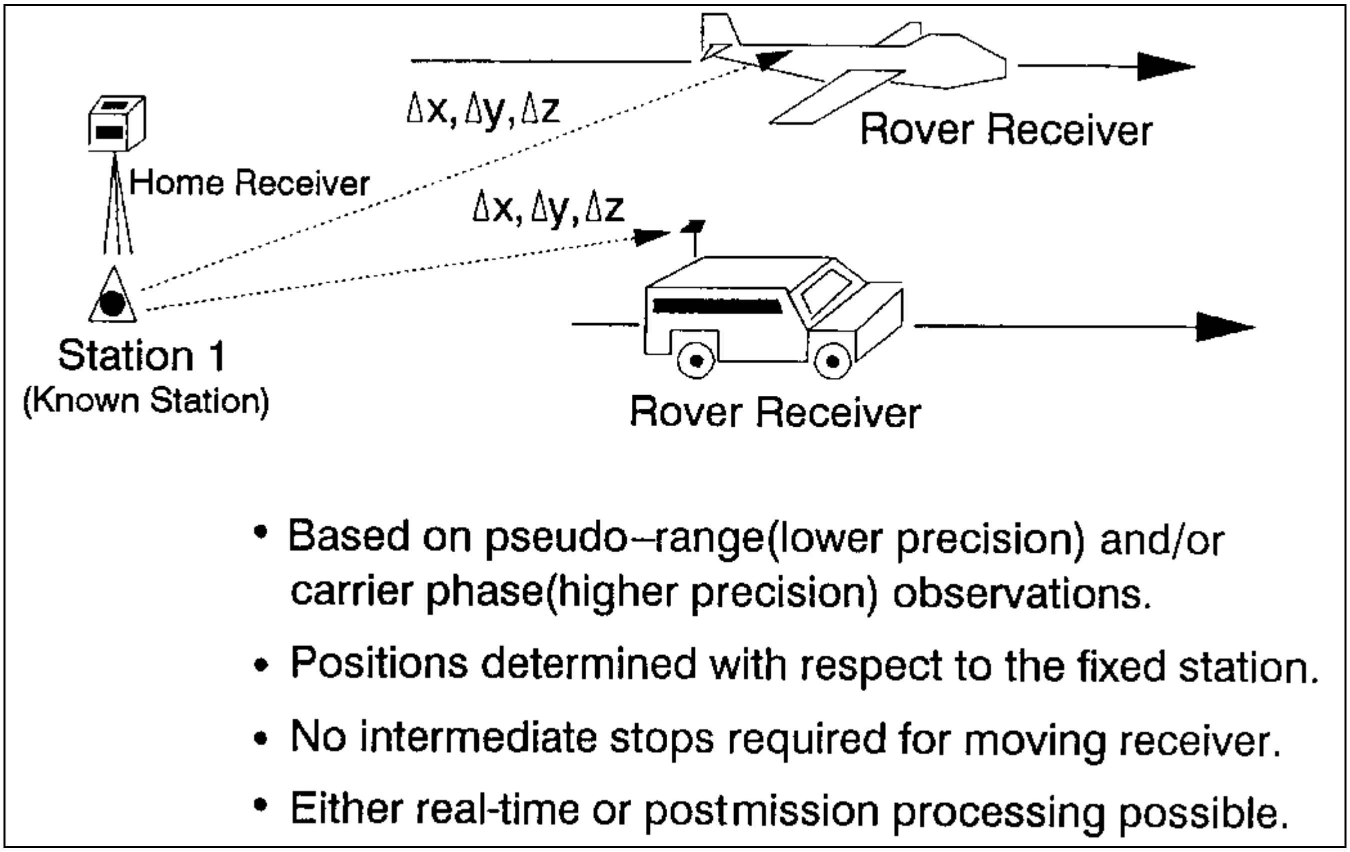


Figure 9-7. Kinematic survey techniques

surveying could achieve centimeter-level accuracy over distances up to 30 km.

9-11. Pseudo-Kinematic GPS Survey Techniques

Pseudo-kinematic GPS surveying is similar to stop-and-go techniques except that loss of satellite lock is tolerated when the receiver is transported between occupation sites (in fact, the roving receiver can be turned off during movement between occupation sites, although this is not recommended). This feature provides the surveyor with a more favorable positioning technique since obstructions such as bridge overpasses, tall buildings, and overhanging vegetation are common. Loss of lock that may result due to these obstructions is more tolerable when pseudo-kinematic techniques are employed.

a. General. The pseudo-kinematic techniques require that one receiver be placed over a known control station. A rover receiver occupies each unknown station for 5 min. Approximately 1 hr after the initial station occupation, the same rover receiver must reoccupy each unknown station.

b. Common satellite requirements. The pseudo-kinematic technique requires that at least four of the same satellites are observed between initial station occupations and the requisite reoccupation. For example, the rover receiver occupies Station A for the first 5 min and tracks satellites 6, 9, 11, 12, 13; then 1 hr later, during the second occupation of Station A, the rover receiver tracks satellites 2, 6, 8, 9, 19. In this example, only satellites 6 and 9 are common to the two sets, so the data cannot be processed because four common satellites were not tracked for the initial station occupation and the requisite reoccupation.

c. Planning. Prior mission planning is essential in conducting a successful pseudo-kinematic survey. Especially critical is the determination of whether or not common satellite coverage will be present for the desired period of the survey. Also, during the period of observation, one receiver, the base receiver, must continuously occupy a known control station.

d. Pseudo-kinematic data processing. Pseudo-kinematic survey satellite data records and resultant baseline processing methods are similar to those performed for static GPS surveys. Since the pseudo-kinematic technique requires each station to be occupied for 5 min and then reoccupied for 5 min approximately an hour later, this technique is not suitable when control stations are widely spaced and transportation between stations within the allotted time is impractical.

e. Accuracy of pseudo-kinematic surveys. Pseudo-kinematic survey accuracies are similar to kinematic survey accuracies of a few centimeters.

9-12. Rapid Static Surveying Procedures

Rapid static surveying is a combination of the stop-and-go kinematic, pseudo-kinematic, and static surveying methods. The rover or remote receiver spends only a short time on each station, loss of lock is allowed between stations, and accuracies are similar to static. However, rapid static surveying does not require re-observation of remote stations like pseudo-kinematic. The rapid static technique does require the use of dual-frequency (L1/L2) GPS receivers with either cross correlation or squaring or any other technique used to compensate for A-S.

a. Survey procedure. Rapid static surveying requires that one receiver be placed over a known control point. A rover or remote receiver occupies each unknown station for 5-20 min, depending on the number of satellites and their geometry. Because most receiver operations are manufacturer-specific, following the manufacturers' guidelines and procedures for this type of survey is important.

b. Rapid static data processing. Data collected in the rapid static mode should be processed in accordance with the manufacturer's specifications. See Chapter 10 for more information on post-processing GPS data.

c. Accuracy of rapid static surveys. Accuracies of rapid static surveys are similar to static surveys of a centimeter or less. This method can be used for medium-to-high accuracy surveys up to 1/1,000,000.

9-13. OTF/RTK Surveying Techniques

OTF/RTK surveying is similar to kinematic differential GPS surveying because it requires two receivers recording observations simultaneously and allows the rover receiver to be moving. Unlike kinematic surveying, OTF/RTK surveying techniques use dual-frequency L1/L2 GPS observations and can handle loss of satellite lock. Since OTF/RTK uses the L2 frequency, the GPS receiver must be capable of tracking the L2 frequency during A-S. There are several techniques used to obtain L2 during A-S. These include the squaring and cross-correlation methods.

a. Ambiguity resolution. As explained before in paragraph 9-7*b*, successful ambiguity resolution is required for successful baseline formulations. The OTF/RTK technology allows the remote to initialize and resolve these integers without a period of static initialization. With OTF/RTK, if loss of satellite lock occurs, initialization can occur while in motion. The integers can be resolved at the rover within 10-30 sec, depending on the distance from the reference station. OTF/RTK uses the L2 frequency transmitted by the GPS satellites in the ambiguity resolution. After the integers are resolved, only the L1 C/A is used to compute the positions.

b. Survey procedure. OTF/RTK surveying requires dual frequency L1/L2 GPS receivers. One of the GPS receivers is set over a known point, and the other is placed on a moving or mobile platform. If the survey is performed in real time, a data link and a processor (external or internal) are needed. The data link is used to transfer the raw data from the reference station to the remote.

(1) Internal processor. If the OTF/RTK system is done with an internal processor (i.e., built into the receiver), follow manufacturer's guidelines.

(2) External processor. If OTF/RTK is performed with external processors (i.e., notebook computer), then computer at the reference (386-based PC) collects the raw GPS data and formats it to be sent via a data link to the remote. The notebook computer at the rover (486/33 based PC) processes the raw data from the reference and remote receivers to resolve the integers and obtain a position.

c. Accuracy of OTF/RTK surveys. OTF/RTK surveys are accurate to within 10 cm when the distance from the reference to the rover does not exceed 20 k. Results of testing by TEC produced results of less then 10 cm.

Chapter 10
Post-processing Differential GPS Observational Data

10-1. General

GPS baseline solutions are usually generated through an iterative process. From approximate values of the positions occupied and observation data, theoretical values for the observation period are developed. Observed values are compared to computed values, and an improved set of positions occupied is obtained using least squares minimization procedures and equations modeling potential error sources.

a. Processing time is dependent on the accuracy required, software development, computer hardware used, data quality, and amount of data. In general, high accuracy solutions, crude computer software and hardware, low-quality data, and high volumes of data will cause longer processing times.

b. The ability to determine positions using GPS is dependent on the effectiveness of the user to determine the range or distance of the satellite from the receiver located on the earth. There are two general techniques currently operational to determine this range: pseudo-ranging and carrier beat phase measurement. These techniques are discussed in further detail below.

c. The user must take special care when attempting a baseline formulation with observations from different GPS receiver manufacturers. It is important to ensure that observables being used for the formulation of the baseline are of a common format (i.e., RINEX). The common data exchange formats required for a baseline formulation exist only between receivers produced by the same manufacturer, but there are some exceptions.

d. This chapter will discuss general post-processing issues. Due to the increasing number and variety of software packages available, consult the manufacturer guidelines when appropriate.

10-2. Pseudo-Ranging

The pseudo-range observable is calculated from observations recorded during a GPS survey. The pseudo-range observable is the difference between the time of signal transmission from the satellite, measured in the satellite time scale, and the time of signal arrival at the receiver antenna, measured in the receiver time scale. When the differences between the satellite and the receiver clocks are reconciled and applied to the pseudo-range observables, the resulting values are corrected pseudo-range values. The value found by multiplying this time difference by the speed of light is an approximation of the true range between the satellite and the receiver, or a true pseudo-range. A more exact approximation of true range between the satellite and receiver can be determined if ionosphere and troposphere delays, ephemeris errors, measurement noise, and unmodeled influences are taken into account while pseudo-ranging calculations are performed. The pseudo-range can be obtained from either the C/A-code or the more precise P-code (if access is available).

10-3. Carrier Beat Phase Observables

The carrier beat phase observable is the phase of the signal remaining after the internal oscillated frequency generated in the receiver is differenced from the incoming carrier signal of the satellite. The carrier beat phase observable can be calculated from the incoming signal or from observations recorded during a GPS survey. By differencing the signal over a period or epoch of time, one can count the number of wavelengths that cycle through the receiver during any given specific duration of time. The unknown cycle count passing through the receiver over a specific duration of time is known as the cycle ambiguity. There is one cycle ambiguity value per satellite/receiver pair as long as the receiver maintains continuous phase lock during the observation period. The value found by measuring the number of cycles going through a receiver during a specific time, when given the definition of the transmitted signal in terms of cycles per second, can be used to develop a time measurement for transmission of the signal. Once again, the time of transmission of the signal can be multiplied by the speed of light to yield an approximation of the range between the satellite and receiver. The biases for carrier beat phase measurement are the same as for pseudo-ranges although a higher accuracy can be obtained using the carrier. A more exact range between the satellite and receiver can be formulated when the biases are taken into account during derivation of the approximate range between the satellite and receiver.

10-4. Baseline Solution by Linear Combination

The accuracy achievable by pseudo-ranging and carrier beat phase measurement in both absolute and relative positioning surveys can be improved through processing that incorporates differencing of the mathematical models of the observables. Processing by differencing takes advantage of correlation of error (e.g., GPS signal, satellite ephemeris, receiver clock, and atmospheric propagation errors) between receivers, satellites, and epochs, or combinations thereof, in order to improve GPS processing. Through differencing, the effects of the errors that are common to the observations being processed are eliminated or at least greatly reduced. Basically, there are three broad processing techniques that incorporate differencing: single differencing, double differencing, and triple differencing. Differenced solutions generally proceed in the following order: differencing between receivers takes place first, between satellites second, and between epochs third.

a. Single differencing. There are three general single differencing processing techniques: between receivers, between satellites, and between epochs (see Figure 10-1).

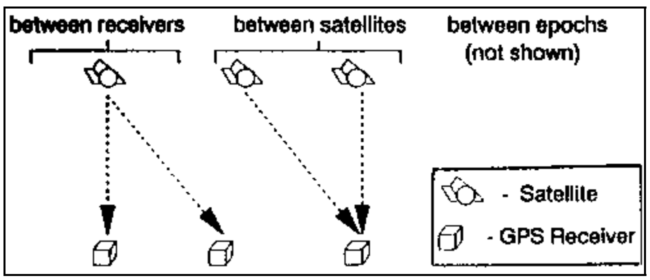


Figure 10-1. Single differencing

(1) Between receivers. Single differencing the mathematical models for a pseudo-range (P- or C/A-code) or carrier phase observable measurements between receivers will eliminate or greatly reduce satellite clock errors and a large amount of satellite orbit and atmospheric delays.

(2) Between satellites. Single differencing the mathematical models for pseudo-range or carrier phase observable measurements between satellites eliminates receiver clock errors. Single differencing between satellites can be done at each individual receiver during observations as a precursor to double differencing and in order to eliminate receiver clock errors.

(3) Between epochs. Single differencing the mathematical models between epochs takes advantage of the Doppler shift or apparent change in the frequency of the satellite signal by the relative motion of the transmitter and receiver. Single differencing between epochs is generally done in an effort to eliminate cycle ambiguities. There are three forms of single differencing techniques between epochs currently in use today: Intermittently Integrated Doppler (IID), Consecutive Doppler Counts (CDC), and Continuously Integrated Doppler (CID). IID uses a technique whereby Doppler count is recorded for a small portion of the observation period, the Doppler count is reset to zero, and then at a later time the Doppler count is restarted during the observation period. CDC uses a technique whereby Doppler count is recorded for a small portion of the observation period, reset to zero, and then restarted immediately and continued throughout the observation period.

b. Double differencing. Double differencing is actually a differencing of two single differences (as detailed in *a* above). There are two general double differencing processing techniques: receiver-time double and receiver-satellite (see Figure 10-2). Double difference processing techniques eliminate clock errors.

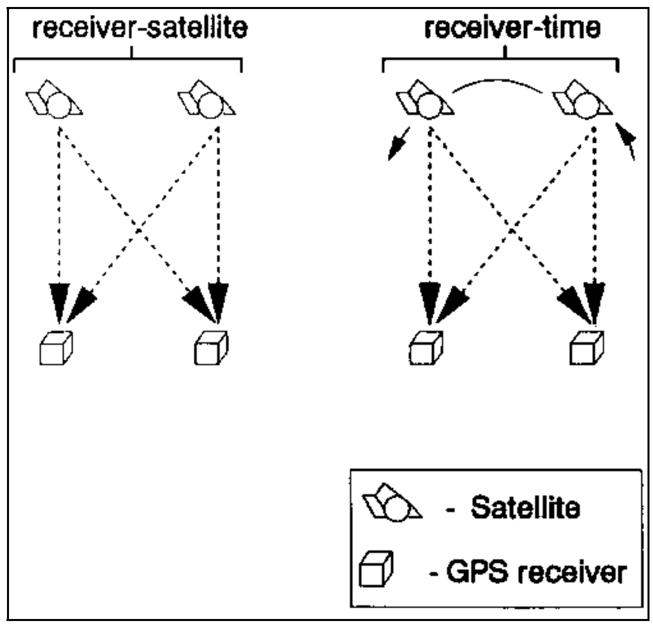


Figure 10-2. Double differencing

(1) Receiver-time double differencing. This technique uses a change from one epoch to the next, in the between-receiver single differences for the same satellite. Using this technique eliminates satellite-dependent integer cycle ambiguities and simplifies editing of cycle slips.

(2) Receiver-satellite double differencing. There are two different techniques that can be used to compute a receiver-satellite double difference. One technique involves using two between-receiver single differences.

This technique also uses a pair of receivers, recording different satellite observations during a survey session and then differencing the observations between two satellites. The second technique involves using two between-satellite single differences. This technique also uses a pair of satellites, but different receivers, and then differences the satellite observations between the two receivers.

c. Triple differencing. There is only one triple differencing processing technique: receiver-satellite-time (see Figure 10-3). All errors eliminated during single- and double-differencing processing are also eliminated during triple differencing. When used in conjunction with carrier beat phase measurements, triple differencing eliminates initial cycle ambiguity. During triple differencing, the data are also automatically edited by the software to delete any data that cannot be solved , so that the unresolved data are ignored during the triple difference solution. This feature is advantageous to the user because of the reduction in the editing of data required; however, degradation of the solution may occur if too much of the data are eliminated during triple differencing.

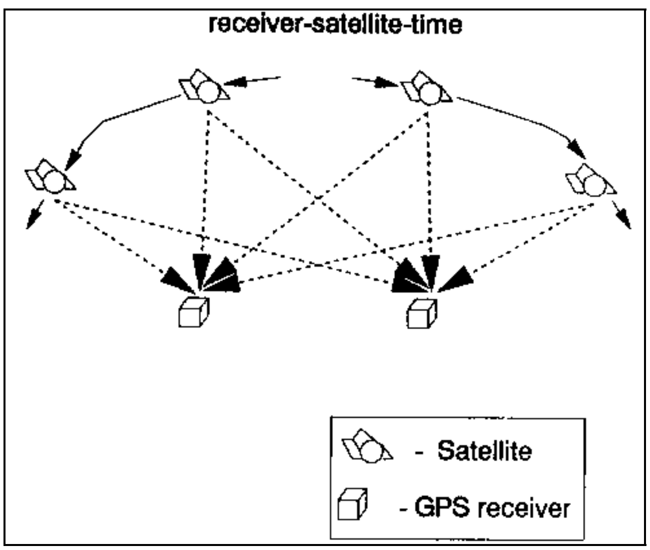


Figure 10-3. Triple differencing

10-5. Baseline Solution by Cycle Ambiguity Recovery

The resultant solution (baseline vector) produced from carrier beat phase observations when differencing resolves cycle ambiguity is called a "fixed" solution. The exact cycle ambiguity does not need to be known to produce a solution; if a range of cycle ambiguities is known, then a "float" solution can be formulated from the range of cycle ambiguities. It is desirable to formulate a fixed solution. However, when the cycle ambiguities cannot be resolved, which occurs when a baseline is between 20 and 65 km in length, a float solution may actually be the best solution. The fixed solution may be unable to determine the correct set of integers (i.e., "fix the integers") required for a solution. Double-differenced fixed techniques can generally be effectively used for positional solutions over short baselines less than 20 km in length. Double differenced float techniques normally can be effectively used for positional solutions for medium-length lines between 20 and 65 km in length.

10-6. Field/Office Data Processing and Verification

a. It is strongly recommended that baselines should be processed daily in the field. This allows the user to identify any problems that may exist. Once baselines are processed, the field surveyor should review each baseline output file. The procedures used in baseline processing are manufacturer-dependent. Certain computational items within the baseline output are common among manufacturers and may be used to evaluate the adequacy of the baseline observation in the field. A list of the triple difference, float double difference, and fixed double difference vectors (*dx-dy-dz*) are normally listed. The geodetic azimuth and distance between the two stations are also listed. The RMS is a quality factor that helps the user determine which vector solution (triple, float, or fixed) to use in an adjustment. The RMS is dependent on the baseline length and the length of time the baseline was observed. Table 10-1 provides guidelines for determining the baseline quality. If the fixed solution meets the criteria in this table, the fixed vector should be used in the adjustment. In some cases the vector passes the RMS test, but after adjustment the vector does not fit into the network. If this occurs, the surveyor should try using the float vector in the adjustments or check to make sure stations were occupied correctly.

b. The first step in data processing is transferring the observation data to a storage device for archiving and/or further processing. Examples of storage devices include a hard disc drive, 5.25-in. disc, 3.5-in. disc, magnetic tape, etc.

c. Once observation data have been downloaded, preprocessing of data can be completed. Pre-processing consists of smoothing/editing the data and ephemeris determination. Smoothing and editing are done to ensure

Table 10-1
Post-processing Criteria

Distance Between Receivers, km	RMS Criteria Formulation (d = distance between receivers)	Formulated RMS Range, cycles	Formulated RMS Range, m
0 - 10	≤(0.02 + (0.004*d))	0.02 - 0.06	0.004 - 0.012
10 - 20	≤(0.03 + (0.003*d))	0.06 - 0.09	0.012 - 0.018
20 - 30	≤(0.04 + (0.0025*d))	0.09 - 0.115	0.018 - 0.023
30 - 40	≤(0.04 + (0.0025*d))	0.115 - 0.14	0.023 - 0.027
40 - 60	≤(0.08 + (0.0015*d))	0.14 - 0.17	0.027 - 0.032
60 - 100	≤0.17	0.17	0.032
> 100	≤0.20	0.20	0.04

Note:

1. These are only general post-processing criteria that may be superseded by GPS receiver/software manufacturer guidelines; consult those guidelines when appropriate.
2. For lines longer than 50 km, dual frequency GPS receivers are recommended to meet these criteria.

data quantity and quality. Activities done during smoothing and editing include determination and elimination of cycle slips; editing gaps in information; and differencing between receivers, satellites, and epochs.

d. Retrieval of post-processed ephemerides may be required depending on the type of receiver used for the survey. Codeless receivers require a post-processed ephemerides file, either that recorded by another GPS receiver concurrent with conduct of the survey or post-processed ephemerides provided by an ephemeris service. Code receivers do not require post-processed ephemerides since they automatically record the broadcast ephemerides during conduct of the survey.

10-7. Post-processing Criteria

Generally, post-processing software will give three solutions: a triple difference, a double-difference fixed solution, and a double-difference float solution. In addition to RDOP as a measurement of the quality of data reduction, methods exist today to gauge the success of an observation session based on data processing done by a differencing process.

a. RMS. RMS is a measurement (in units of cycles or meters) of the quality of the observation data collected during a point in time. RMS is dependent on line length, observation strength, ionosphere, troposphere, and multipath. In general, the longer the line and the more signal interference by other electronic gear, ionosphere, troposphere, and multipath, the higher the RMS will be. A good RMS factor (one that is low, e.g., between 0.01 and 0.2 cycles) may not always indicate good results but is one indication to be taken into account. RMS can generally be used to judge the quality of the data used in the post-processing and the quality of the post-processed baseline vector.

b. Repeatability. Redundant lines should agree to the level of accuracy that GPS is capable of measuring to. For example, if GPS can measure a 10-km baseline to 1 cm ± 1 ppm, the expected ratio of misclosure would be

$$\frac{0.01 \text{ m} + 0.01\text{m}}{10{,}000} = 1{:}500{,}000$$

Repeated baselines should be near the corresponding

$$\frac{1 \text{ cm} + 1 \text{ ppm}}{\text{baseline}}$$

ratio. See Table 10-2 for an example of repeatability of GPS baselines.

c. Other general information included in a baseline solution.

(1) The following information is typically output from a baseline solution:

(a) Listing of the filename.

(b) Types of solutions (single, double, or triple difference).

Table 10-2
Example of Repeatability of GPS Baselines

Baseline	X	Y	Z	Distance
Line 1	5,000.214	4,000.000	7,680.500	9,999.611
Line 2	5,000.215	4,000.005	7,680.491	9,999.607
Difference	0.001	0.005	0.009	
Ratio = 0.010 / 9,999.6	= 1:967,000			

(c) Satellite availability during the survey for each station occupied.

(d) Ephemeris file used for the solution formulation.

(e) Type of satellite selection (manual or automatic).

(f) Elevation mask.

(g) Minimum number of satellites used.

(h) Meteorological data (pressure, temperature, humidity).

(i) Session time (date, time).

(j) Data logging time (start, stop).

(k) Station information: location (latitude, longitude, height), receiver serial number used, antenna serial number used, ID numbers, antenna height.

(l) RMS.

(m) Solution files: Δx, Δy, Δz between stations, slope distance between stations, Δlatitude, Δlongitude between stations, distance between stations, and Δheight.

(n) Epoch intervals.

(o) Number of epochs.

(2) Sample static baseline formulations from two equipment manufacturers, Ashtech, Inc., (GPPS) and Trimble Navigation (GPSurvey), are shown in Figures 10-4 and 10-5, respectively. The baseline formulations have been annotated with the conventions in (a)-(o) above as an aid in an explanation of the results.

10-8. Field/Office Loop Closure Checks

Post-processing criteria are aimed at an evaluation of a single baseline. In order to verify the adequacy of a group of connected baselines, one must perform a loop closure on the baselines formulated. When GPS baseline traverses or loops are formed, their linear (internal) closure should be determined in the field. If job requirements are less than Third-Order (1:10,000 or 1:5,000), and the internal loop/traverse closures are very small, a formal (external) adjustment may not be warranted.

a. Loop closure software packages. The internal closure determines the consistency of the GPS measurements. Internal closures are applicable for loop traverses and GPS networks. It is required that one baseline in the loop be independent. An independent baseline is observed during a different session or different day. Today, many of the better post-processing software packages come with a loop closure program. Refer to the individual manufacturer post-processing user manuals for a discussion on the particulars of the loop closure program included with the user hardware.

b. General loop closure procedure. If the user post-processing software package does not contain a loop closure program, the user can perform a loop closure as shown below.

(1) List the Δx, Δy, and Δz and length of the baseline being used in a table of the form shown in Table 10-3.

(2) Sum the Δx, Δy, Δz, and distance components for all baselines used in the loop closure. For instance, for the baselines in Table 10-3, the summation would be $\Sigma\Delta x$, $\Sigma\Delta y$, $\Sigma\Delta z$, and Σ*Distances* or $(\Delta x\#1 + \Delta x\#2 + \Delta x\#3)$, $(\Delta y\#1 + \Delta y\#2 + \Delta y\#3)$, $(\Delta z\#1 + \Delta z\#2 + \Delta z\#3)$, and $(\Delta Distance\#1 + \Delta Distance\#2 + \Delta Distance\#3)$, respectively.

(3) Once summation of the Δx, Δy, Δz, and Δ*Distance* components has been completed, the square of each of the summations should be added together and the square root of this sum then taken. This resultant value is the misclosure vector for the loop. This relationship can be expressed in the following manner:

```
Ashtech, Inc. GPPS-L           Program: LINECOMP        Version: 4.5.00
                            Tue Jan 25 10:16:25 1994
_______________________________________________________________________
Project information
GPS Survey                |25-character project name  [ The | is in column 26. ]
3203C                     |5-character session name
Project information

Known-station parameters
00                        |Receiver identifier used in "LOGTIMES" file
000000                    |Project station number
MANT                      |4-character short name
FIXED STATION             |25-character long name
564 270 DCO PIC__________ |25-character comment field
0                         |Position extraction (0=below,1=U-file,2=proj. file)
N  40  2 18.36587         |Latitude deg-min-sec  (g=good;b=bad)
E 285 56 49.57251         |E-Longitude deg-min-sec (g=good;b=bad)
W  74  3 10.42749         |W-Longitude deg-min-sec (g=good;b=bad)
   -12.0807               |Ellipsoidal height  (m) (g=good;b=bad)
     0.0000               |North antenna offset(m)
     0.0000               |East antenna offset (m)
  1.4300  0.0000  0.0000  |Vert antenna offset (m): slant/radius/added_offset
    20.0                  |Temperature (degrees C)
    50.0                  |Humidity (percent)
  1010.0                  |Pressure (millibars)
UMANTC93.320              |Measurement filename (restricted to 24 characters)
Known-station parameters

Unknown-station parameters
00                        |Receiver identifier used in "LOGTIMES" file
000000                    |Project station number
FTM1                      |4-character short name
UNKNOWN STATION           |25-character long name
564 270 DCO PIC__________ |25-character comment field
0                         |Position extraction (0=below,1=U-file,2=proj. file)
N  40 18 45.82336         |Latitude deg-min-sec  (g=good;b=bad)
E 285 57 46.72853         |E-Longitude deg-min-sec (g=good;b=bad)
W  74  2 13.27147         |W-Longitude deg-min-sec (g=good;b=bad)
   -20.5991               |Ellipsoidal height  (m) (g=good;b=bad)
     0.0000               |North antenna offset(m)
     0.0000               |East antenna offset (m)
  0.0000  0.0000  0.0000  |Vert antenna offset (m): slant/radius/added_offset
    20.0                  |Temperature (degrees C)
    50.0                  |Humidity (percent)
  1010.0                  |Pressure (millibars)
UFTM1C93.320              |Measurement filename (restricted to 24 characters)
Unknown-station parameters

Run-time parameters
   10                     |First epoch to process
   -1                     |Final epoch to process (-1 = last available)
15.0                      |Elevation cutoff angle (degrees)
1                         |Data to process (0=Wdln;1=L1;2=L2;3=Llc;6=RpdSt)
0.010000                  |Convergence criterion (meters)
00 00 00 00 00 00 00      |Omit these satellites (up to 7)
10                        |Maximum iterations for tlsq and dlsq
00 00 00 00 00 00 00      |Forbidden reference SVs (up to 7)
yes                       |Apply tropo delay correction
Run-time parameters
```

Figure 10-4. Sample static baseline formulation (Ashtech, Inc., GPPS-L) (Sheet 1 of 5)

```
LINECOMP 4.5.00 12/11/92

FIXED U-File from P-Code receiver.
UNKWN U-File from P-Code receiver.

FIXED U-File used BROADCAST orbits.
UNKWN U-File used BROADCAST orbits.

Common start of two UFILES: 1993/11/16 20:23:60.00
Common end   of two UFILES: 1993/11/16 22:00:20.00
    Selected first epoch:  10
    Selected last  epoch:  290
For SV  1 there are   280 triple-difference measurements.
For SV  5 there are   181 triple-difference measurements.
For SV 12 there are   136 triple-difference measurements.
For SV 15 there are   152 triple-difference measurements.
For SV 20 there are   181 triple-difference measurements.
For SV 21 there are   181 triple-difference measurements.
For SV 23 there are   181 triple-difference measurements.
For SV 25 there are   181 triple-difference measurements.
Epoch interval (seconds): 20.000000


THE TRIPLE DIFFERENCE SOLUTION (L1)
Measure of geometry: 0.712832
num_meas =  1192        num_used =  1191         rms_resid = 0.002725(m)
Post-Fit Chisq =  1403.765            NDF      =     11.028


  Sigmax (m):       0.347912
  Sigmay (m):       0.646995
  Sigmaz (m):       0.327369
  x     y     z
x 1.00
y 0.17y 1.00
z 0.12z-0.50z 1.00

del_station: -0.000007 -0.000001 0.000027
     Station1: FIXED STATION                 Station2: UNKNOWN STATION
          (00000)    (MANT)                       (00000)    (FTM1)
 Latitude:  40.03843496  40  2 18.36587          40.31281330  40 18 46.12789
 E-Long  : 285.94710348 285 56 49.57251         285.96293196 285 57 46.55506
 W-Long  :  74.05289652  74  3 10.42749          74.03706804  74  2 13.44494
 E-Height: -12.0807                               -2.8736

 Baseline vector:     -4104.5950     19261.5243     23284.3880

 Mark1_xyz  :  1343513.8259  -4701767.9098   4081246.0717
 Az1 El1 D1 :        2.52867       -0.1200     30496.1759
 E1 N1 U1   :     1350.8948     30465.6429        9.2071
 Mark2_xyz  :  1339409.2309  -4682506.3855   4104530.4598
 Az2 El2 D2 :      182.53888       -0.1546     30496.1759
 E2 N2 U2   :    -1345.4669    -30467.1353       -9.2071

Double-Difference Epochs:
  Prn:   1  Start epoch:   11  End epoch:  290
  Prn:   5  Start epoch:  110  End epoch:  290
  Prn:  12  Start epoch:  110  End epoch:  249
  Prn:  15  Start epoch:  139  End epoch:  290
  Prn:  20  Start epoch:  110  End epoch:  290
```

Figure 10-4. (Sheet 2 of 5)

```
    Prn:  21  Start epoch:  110  End epoch:  290
    Prn:  23  Start epoch:  110  End epoch:  290
    Prn:  25  Start epoch:  110  End epoch:  290


  THE FLOAT DOUBLE DIFFERENCE SOLUTION (L1)
  Measure of geometry: 0.195687     Wavelength = 0.190294 (m/cycle)
  num_meas =  1200        num_used =  1200        rms_resid = 0.013991(m)
  Post-Fit Chisq =   186.429              NDF       =    11.111

  Reference SV:  1
    SV     Ambiguity        FIT    Meas      SV     Ambiguity        FIT    Meas
     5        59386.483f  0.054    182       12      -1227312.585f  0.050    138
    15      2121069.816f  0.097    152       20        531426.734f  0.072    182
    21      -184904.908f  0.073    182       23      -1075927.194f  0.062    182
    25       646212.381f  0.093    182

    Sigmax (m):       0.049793
    Sigmay (m):       0.056987
    Sigmaz (m):       0.026423
    SigmaN (cy):      0.283527
    SigmaN (cy):      0.289386
    SigmaN (cy):      0.245180
    SigmaN (cy):      0.217266
    SigmaN (cy):      0.134735
    SigmaN (cy):      0.204750
    SigmaN (cy):      0.196954
    x     y     z     N     N     N     N     N     N     N
  x 1.00
  y 0.19y 1.00
  z 0.08z-0.30z 1.00
  N 0.77N 0.74N-0.23N 1.00
  N 0.53N 0.90N-0.22N 0.92N 1.00
  N-0.81N 0.35N-0.35N-0.27N 0.01N 1.00
  N 0.87N 0.27N-0.35N 0.80N 0.58N-0.57N 1.00
  N 0.39N-0.52N-0.30N 0.04N-0.24N-0.51N 0.55N 1.00
  N 0.70N 0.11N-0.56N 0.62N 0.39N-0.47N 0.91N 0.71N 1.00
  N-0.68N-0.57N-0.38N-0.71N-0.70N 0.41N-0.40N 0.35N-0.09N 1.00

  del_station: -0.000000 -0.000000 0.000000
       Station1: FIXED STATION                Station2: UNKNOWN STATION
                 (00000)     (MANT)                     (00000)     (FTM1)
   Latitude:  40.03843496  40  2 18.36587           40.31281268  40 18 46.12563
   E-Long  : 285.94710348 285 56 49.57251          285.96293166 285 57 46.55396
   W-Long  :  74.05289652  74  3 10.42749           74.03706834  74  2 13.44604
   E-Height: -12.0807                                -2.8299

   Baseline vector:     -4104.5984     19261.4419     23284.3633

   Mark1_xyz  :  1343513.8259  -4701767.9098   4081246.0717
   Az1 El1 D1 :        2.52863        -0.1199     30496.1054
   E1 N1 U1   :      1350.8687     30465.5734         9.2508
   Mark2_xyz  :  1339409.2275  -4682506.4679   4104530.4350
   Az2 El2 D2 :      182.53884        -0.1547     30496.1054
   E2 N2 U2   :     -1345.4410    -30467.0660        -9.2508


  AMBIGUITY RESOLUTION
                          1          2          3          4
  Abs Contrast          0.000      0.000      0.000      0.000
```

Figure 10-4. (Sheet 3 of 5)

```
Contrast                              99.999      100.000      100.000
Change Chi2            318.829       907.189     1231.184     1556.459
Bias S 1: 5              59387         59385        59387        59387
Bias S 1:12           -1227312      -1227314     -1227312     -1227312
Bias S 1:15            2121070       2121070      2121069      2121071
Bias S 1:20             531427        531426       531427       531427
Bias S 1:21            -184905       -184905      -184905      -184905
Bias S 1:23           -1075927      -1075928     -1075927     -1075927
Bias S 1:25             646212        646213       646212       646213
NDF=127.0000 Chi2=186.4289
                             1             2            3            4
Abs Contrast             0.000         0.000        0.000        0.000
Contrast                              99.999      100.000      100.000
Change Chi2            298.148       843.456     1086.524     1100.925
Bias S 1:12           -1227312      -1227314     -1227313     -1227313
Bias S 1:15            2121070       2121070      2121069      2121071
Bias S 1:20             531427        531426       531427       531426
Bias S 1:21            -184905       -184905      -184905      -184905
Bias S 1:23           -1075927      -1075928     -1075927     -1075928
Bias S 1:25             646212        646213       646212       646213
NDF=127.0000 Chi2=186.4289
                             1             2            3            4
Abs Contrast             0.004         0.000        0.000        0.000
Contrast                              99.986      100.000      100.000
Change Chi2            190.078       526.018      746.284     1076.670
Bias S 1:15            2121070       2121069      2121070      2121069
Bias S 1:20             531427        531427       531426       531426
Bias S 1:21            -184905       -184905      -184905      -184905
Bias S 1:23           -1075927      -1075927     -1075928     -1075928
Bias S 1:25             646212        646212       646213       646212
NDF=127.0000 Chi2=186.4289
                             1             2            3            4
Abs Contrast             4.563         0.000        0.000        0.000
Contrast                             100.000      100.000      100.000
Change Chi2            128.751      2529.042     3851.923     5153.774
Bias S 1: 5              59387         59388        59387        59387
Bias S 1:12           -1227312      -1227311     -1227311     -1227313
NDF=132.0000 Chi2=376.5065


THE FIXED DOUBLE DIFFERENCE SOLUTION (L1)
Measure of geometry: 0.038900     Wavelength = 0.190294 (m/cycle)
num_meas =  1200        num_used =  1188          rms_resid = 0.021554(m)
Post-Fit Chisq =   435.849            NDF       =     11.000

Reference SV:  1                    Integer Search Ratio =    99.986
  SV      Ambiguity         FIT    Meas       SV      Ambiguity          FIT     Meas
   5        59387.000X    0.066    182        12      -1227312.000X    0.070    138
  15      2121070.000X    0.195    140        20        531427.000X    0.065    182
  21      -184905.000X    0.067    182        23      -1075927.000X    0.080    182
  25       646212.000X    0.176    182

  Sigmax (m):       0.009106
  Sigmay (m):       0.015190
  Sigmaz (m):       0.016909
  x      y      z
x 1.00
y-0.37y 1.00
z 0.40z-0.71z 1.00
```

Figure 10-4. (Sheet 4 of 5)

```
del_station: 0.001087 -0.002400 0.000191
     Station1: FIXED STATION                  Station2: UNKNOWN STATION
                (00000)    (MANT)                      (00000)    (FTM1)
 Latitude:  40.03843496  40  2 18.36587          40.31281315  40 18 46.12733
 E-Long  : 285.94710348 285 56 49.57251         285.96293257 285 57 46.55727
 W-Long  :  74.05289652  74  3 10.42749          74.03706743  74  2 13.44273
 E-Height: -12.0807                                -2.9282

 Baseline vector:     -4104.5533     19261.5680     23284.3397

 Mark1_xyz  :  1343513.8259  -4701767.9098   4081246.0717
 Az1 El1 D1 :        2.52877        -0.1201     30496.1610
 E1 N1 U1   :     1350.9471     30465.6258         9.1525
 Mark2_xyz  :  1339409.2726  -4682506.3418   4104530.4115
 Az2 El2 D2 :      182.53898        -0.1545     30496.1610
 E2 N2 U2   :    -1345.5190    -30467.1180        -9.1525
Tue Jan 25 10:18:17 1994
```

Figure 10-4. (Sheet 5 of 5)

Project Name:	ftm1
Processed:	Tuesday, January 25, 1994 11:17
	WAVE Baseline Processor, version 1.01
Summary Reference Index:	1
Fixed Station:	MANT
Antenna Height (meters):	1.430 [True Vertical]
Data file:	MANT320C.DAT
Floating Station:	FTM1
Antenna Height (meters):	0.000 [True Vertical]
Data file:	FTM1320C.DAT
Start Time:	11/16/93 20:21:40 GPS (723 246100)
Stop Time:	11/16/93 22:00:20 GPS (723 252020)
Occupation Time:	0 01:38:40
Measurement Epoch Interval (seconds):	20.00
Solution Type:	Receiver/satellite double difference
	Fixed integer phase ambiguity
	Iono free carrier phase
Solution Acceptability:	Passed
Number of Observations / Number Rejected:	1838 / 0 (0.00% of Total Observations)
Baseline Slope Distance (meters):	30496.196

	Forward	Backward
Normal Section Azimuth:	2° 31' 42.850578"	182° 32' 19.610607"
Vertical Angle:	-0° 07' 12.582816"	-0° 09' 16.140268"

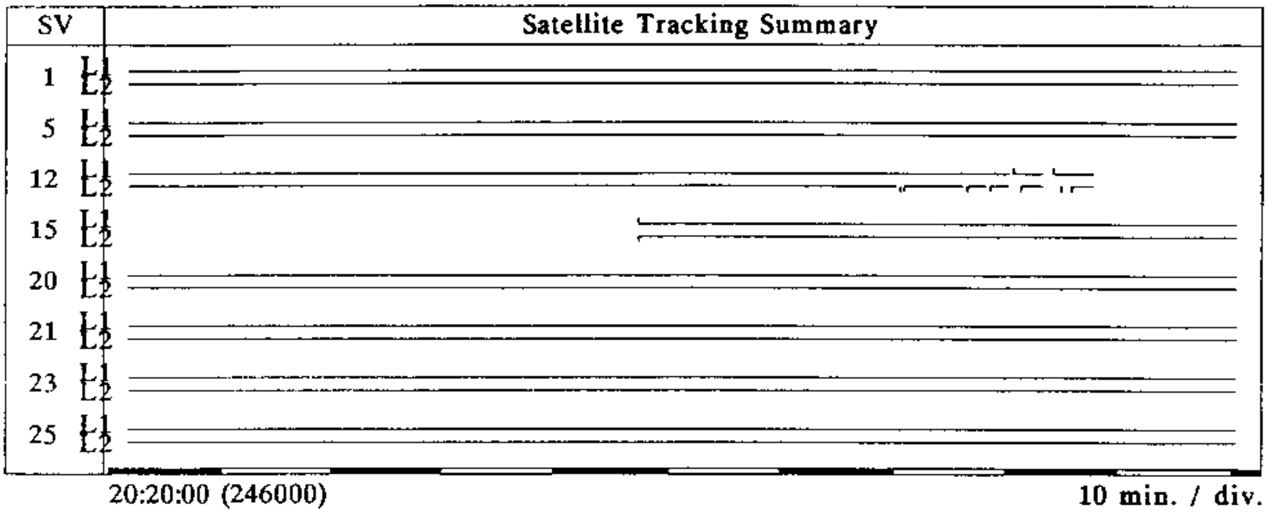
Baseline Components (meters):	dn 30466.437	de 1345.414	du -63.957
	dx -4104.555	dy 19261.587	dz 23284.370
Standard Deviations:	5.303799E-004	9.044810E-004	8.225305E-004
Aposteriori Covariance Matrix:	2.813028E-007		
	-2.038846E-007	8.180858E-007	
	1.759316E-007	-4.827601E-007	6.765565E-007

Reference Variance:	0.633
Variance Ratio 2nd Best/Best Ambiguity Candidate:	28.0
RMS (meters):	0.014

Figure 10-5. Sample static baseline formulation (Trimble Navigation (GP Survey) (Sheet 1 of 3)

Project: ftm1
Processed: Tuesday, January 25, 1994 11:17 WAVE 1.01

Fixed Sta
Position: 40■ 02' 18.244439" N 74■ 03' 11.
X= 1343486.892 Y= -4701771.345



Float Sta
Position: 40■ 18' 46.008533" N 74■ 02' 14.
X= 1339382.336 Y= -4682509.759

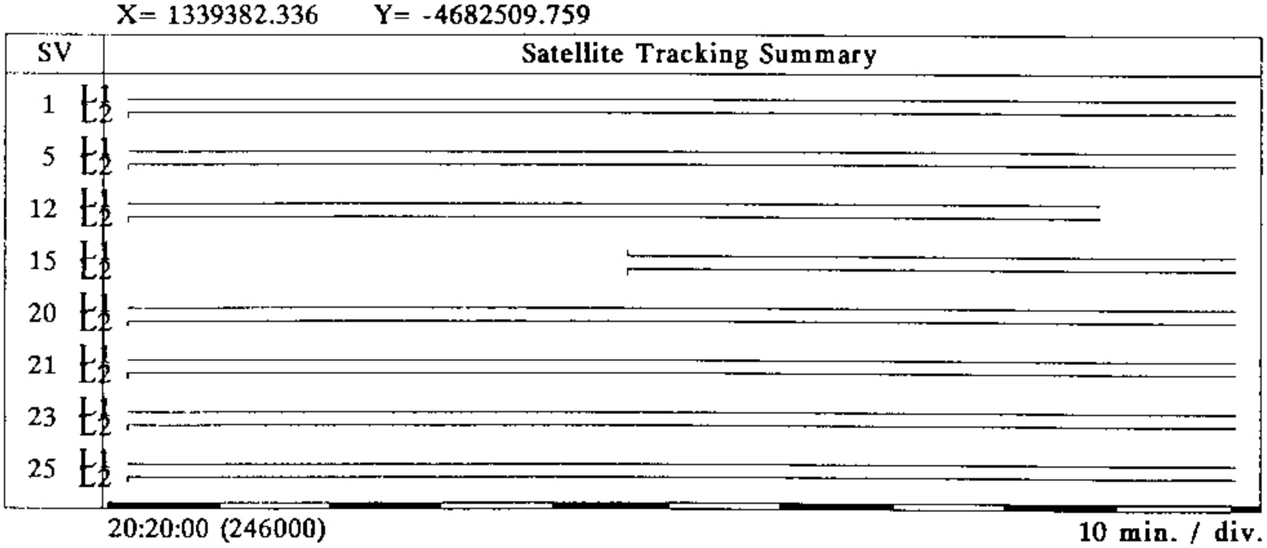


Figure 10-5. (Sheet 2 of 3)

Project: ftm1
Processed: Tuesday, January 25, 1994 11:17 WAVE 1.01

Station si
Shading i

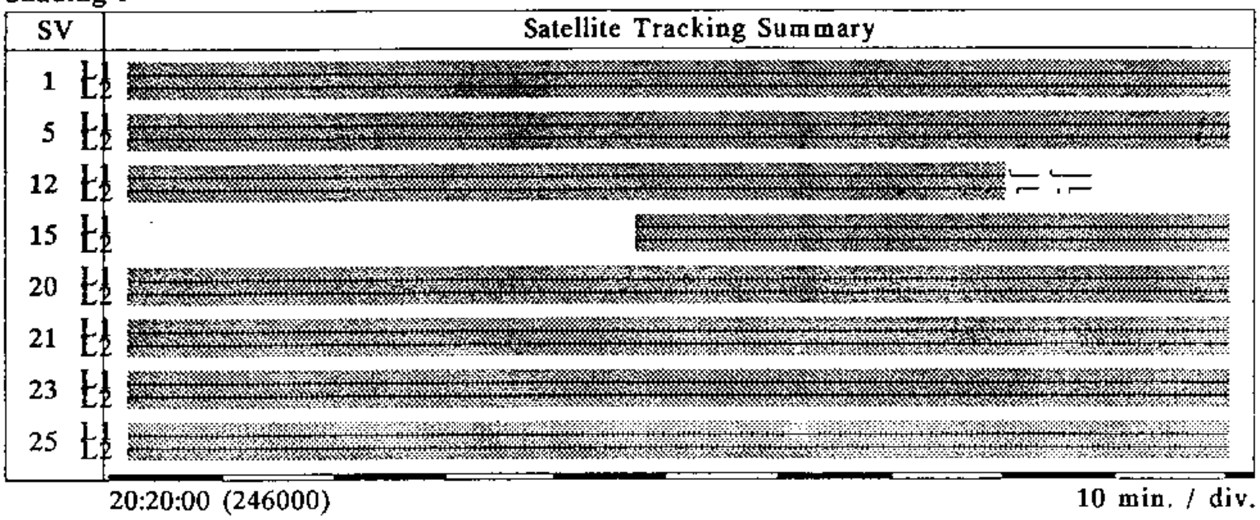


Figure 10-5. (Sheet 3 of 3)

Table 10-3
Loop Closure Procedure

Baseline	Julian Day	Session	Δx	Δy	Δz	ΔDistance
#1	Day	#	Δx #1	Δy #1	Δz #1	Distance #1
#2	Day	#	Δx #2	Δy #2	Δz #2	Distance #2
#3	Day	#	Δx #3	Δy #3	Δz #3	Distance #3

$$m = \sqrt{(\Sigma\Delta x^2) + (\Sigma\Delta y^2) + (\Sigma\Delta z^2)} \tag{10-1}$$

where

m = misclosure for the loop

$\Sigma\Delta x$ = sum of all Δx vectors for baselines used

$\Sigma\Delta y$ = sum of all Δy vectors for baselines used

$\Sigma\Delta z$ = sum of all Δz vectors for baselines used

(4) The loop misclosure ratio may be calculated as follows:

$$Loop\ misclosure\ ratio = \frac{m}{L} \tag{10-2}$$

where

L = total loop distance (perimeter distance)

(5) The resultant value can be expressed in the following form:

1: Loop Misclosure Ratio

with all units for the expressions being in terms of the units used in the baseline formulations (e.g., m, ft, mm, etc.).

c. Sample loop closure computation. Figure 10-6 shows two loops which consist of four stations. During session A on day 065, three GPS receivers observed the baselines between stations 01, 02, and 03 for approximately 1 hr. The receivers were then turned off and the receiver at station 01 was moved to station 04. The tripod heights at stations 02 and 03 were adjusted. The baselines between stations 02, 03, and 04 were then observed during session B, day 065. Stations 01 and 04 were known control stations. This provided an independent baseline for both loops.

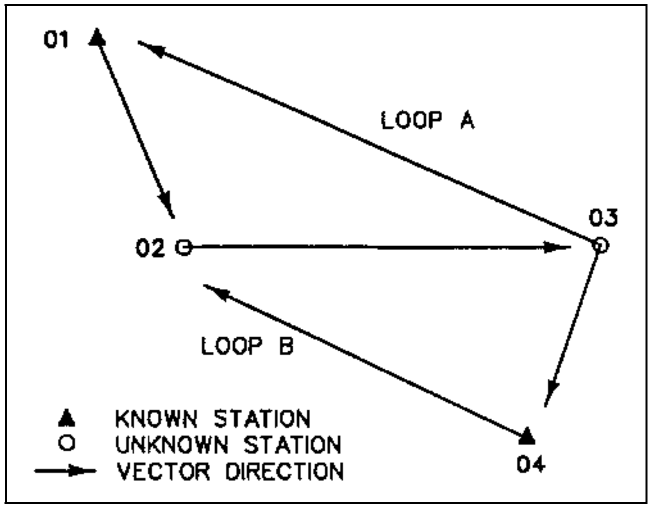


Figure 10-6. Internal loop closure diagram

(1) The closure for loop 01-02-03 is computed with the vectors 01-02 and 01-03, day 065, session A, and the vector 02-03, day 065, session B. The vector 02-03 from session B provides an independent baseline. The loop closure is determined by arbitrarily assigning coordinate values of zero to station 01 (X=0, Y=0, Z=0). The vector from 01-02 is added to the coordinates of station 01. The vector from 02-03, session B, is added to the derived coordinates of station 02. The vector from 03-01 is then added to the station coordinates of 02. Since the starting coordinates of station 01 were arbitrarily chosen as zero, the misclosure is then the computed coordinates of Station 04 (*dx*, *dy*, *dz*). The vector data are listed in Table 10-4.

(2) To determine the relative loop closure, the square root of the sum of the squares of the loop misclosures (*mx*, *my*, *mz*) is divided into the perimeter length of the loop:

Table 10-4
Vector Data for Stations 01, 02, and 03

Baseline	Julian Day	Session	ΔX	ΔY	ΔZ	ΔDistance
01-02	065	A	-4077.865	-2877.121	-6919.829	8531.759
02-03	065	B	7855.762	-3129.673	688.280	8484.196
03-01	065	A	-3777.910	6006.820	6231.547	9443.869

$$\textit{Loop misclosure ratio} = \frac{\left(\Delta x^2 - \Delta y^2 - \Delta z^2\right)^{0.5}}{L} \qquad (10\text{-}3)$$

Where the PD = distance 01-02 + distance 02-03 + distance 03-01, or:

PD = 8531.759 + 8484.196 + 9443.869

= 26,459.82

And where distance 03-01 is computed from:

$(-3777.91^2 + 6006.820^2 + 6231.547^2)^{0.5}$

= 9443.869

(Other distances are similarly computed.)

Summing the misclosures in each coordinate:

Δx = -4077.865 + 7855.762 - 3777.910 = -0.0135

Δy = -2877.121 - 3129.673 + 6006.820 = +0.0264

Δz = -6919.829 + 688.280 + 6231.547 = -0.0021

then

$(\Delta x^2 + \Delta y^2 + \Delta z^2)^{0.5} = 0.029$

Loop misclosure ratio = 0.029/26,459.82

or (approximately) 1 part in 912,000 (1:912,000)

(3) This example is quite simplified; however, it illustrates the necessary mechanics in determining internal loop closures. The values *DX*, *DY*, and *DZ* are present in the baseline output files. The perimeter distance is computed by adding the distances between each point in the loop.

d. External closures. External closures are computed in a similar manner to internal loops. External closures provide information on how well the GPS measurements conform to the local coordinate system. Before the closure of each traverse is computed, the latitude, longitude, and ellipsoid height must be converted to geocentric coordinates (X,Y,Z), using the algorithms given in Chapter 11. If the ellipsoid height is not known, geoid modeling software can be used with the orthometric height to get an approximate ellipsoid height. The external closure will aid the surveyor in determining the quality of the known control and how well the GPS measurements conform to the local network. If the control stations are not of equal precision, the external closures will usually reflect the lower order station. If the internal closure meets the requirements of the job, but the external closure is poor, the surveyor should suspect that the known control is deficient and an additional known control point should be tied into the system.

10-9. Data Management (Archival)

The raw data are defined as data recorded during the observation period. Raw data shall be stored on an appropriate medium (floppy disk, portable hard drive, magnetic tape, etc.). The raw data and the hard copy of the baseline reduction (resultant baseline formulations) shall be stored at the discretion of each USACE Command.

10-10. Flow Diagram

When processing GPS observational data, the progress should generally follow the path shown in Figure 10-7.

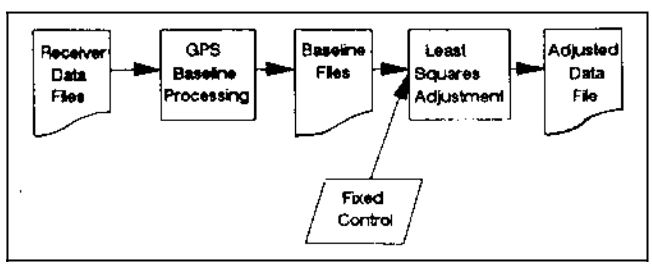


Figure 10-7. GPS data processing flowchart

Chapter 11
Adjustment of GPS Surveys

11-1. General

Differential carrier phase GPS survey observations are adjusted no differently from conventional surveys. Each three-dimensional GPS baseline vector is treated as a separate distance observation and adjusted as part of a trilateration network. A variety of techniques may be used to adjust the observed GPS baselines to fit existing control. Since GPS survey networks often contain redundant observations, they are usually (but not always) adjusted by some type of rigorous least squares minimization technique. This chapter describes some of the methods used to perform horizontal GPS survey adjustments and provides guidance in evaluating the adequacy and accuracy of the adjustment results.

11-2. GPS Error Measurement Statistics

In order to understand the adjustment results of a GPS survey, some simple statistical terms should be understood.

a. Accuracy. Accuracy is how close a measurement or a group of measurements are in relation to a "true" or "known" value.

b. Precision. Precision is how close a group or sample of measurements are to each other. For example, a low standard deviation indicates high precision. It is important to understand that a survey or group of measurements can have a high precision but a low accuracy (i.e., measurements are close together but not close to the known or true value).

c. Standard deviation. The standard deviation is a range of how close the measured values are from the arithmetic average. A low standard deviation indicates that the observations or measurements are close together.

11-3. Adjustment Considerations

a. This chapter deals primarily with the adjustment of horizontal control established using GPS observations. Although vertical elevations are necessarily carried through the baseline reduction and adjustment process, the relative accuracy of these elevations is normally inadequate for engineering and construction purposes. Special techniques and constraints are necessary to determine approximate orthometric elevations from relative GPS observations, as was covered in Chapter 6.

b. The baseline reduction process (described in Chapter 10) directly provides the raw relative position coordinates which are used in a 3D GPS network adjustment. In addition, and depending on the manufacturer's software, each reduced baseline will contain various orientation parameters, covariance matrices, and cofactor and/or correlation statistics which may be used in weighing the final network adjustment. Most least squares adjustments use the accuracy or correlation statistics from the baseline reductions; however, other weighing methods may be used in a least squares or approximate adjustment.

c. The adjustment technique employed (and time devoted to it) must be commensurate with the intended accuracy of the survey, as defined by the project requirements. Care must be taken to prevent the adjustment process from becoming a project in itself.

d. There is no specific requirement that a rigorous least squares type of adjustment be performed on USACE surveys, whether conventional, GPS, or mixed observations. Traditional approximate adjustment methods may be used in lieu of least squares and will provide comparable practical accuracy results.

e. Commercial software packages designed for higher order geodetic densification surveys often contain a degree of statistical sophistication which is unnecessary for engineering survey control densification (i.e., Second-Order or less). For example, performing repeated chi-square statistical testing on observed data intended for 1:20,000 base mapping photogrammetric control may be academically precise but, from a practical engineering standpoint, is inappropriate. The distinction between geodetic surveying and engineering surveying must be fully considered when performing GPS survey adjustments and analyzing the results thereof.

f. Connections and adjustments to existing control networks, such as the NGRS, must not become independent projects. It is far more important to establish dense and accurate local project control than to consume resources tying into First-Order NGRS points miles from the project. Engineering, construction, and property/boundary referencing requires consistent local control with high relative accuracies; accurate connections/references to distant geodetic datums are of secondary importance. (Exceptions might involve projects in support of military operations.) The advent of GPS surveying technology has provided a cost-effective means of tying previously poorly connected USACE projects to the NGRS, and simultaneously transforming the project to the newly defined NAD 83. In performing (adjusting) these connections, care must be taken not to distort or warp long-established project construction/boundary reference points.

11-4. Survey Accuracy

a. General. The accuracy of a survey (whether performed using conventional or GPS methods) is a measure of the difference between observed values and the true values (coordinates, distance, angle, etc.). Since the true values are rarely known, only estimates of survey accuracy can be made. These estimates may be based on the internal observation closures, such as on a loop traverse, or connections with previously surveyed points assumed to have some degree of reliability. The latter case is typically a traverse (GPS or conventional) between two previously established points, either existing USACE project control or the published NGRS network.

(1) GPS internal accuracies are typically far superior to most previously established control networks (including the NAD 83 NGRS). Therefore, determining the accuracy of a GPS survey based on misclosures with external points is not always valid unless statistical accuracy estimates (i.e., station variance-covariance matrices, distance/azimuth relative accuracy estimates, etc.) from the external network's original adjustment are incorporated into the closure analysis for the new GPS work. Such refinements are usually unwarranted for most USACE work.

(2) Most survey specifications and standards (including USACE) classify accuracy as a function of the resultant relative accuracy between two usually adjacent points in a network. This resultant accuracy is estimated from the statistics in an adjustment, and is defined by the size of a 2D or 3D relative error ellipse formed between the two points. Relative distance, azimuth, or elevation accuracy specifications and classifications are derived from this model, and are expressed either in absolute values (e.g., ±1.2 cm or ±3.5 in.) or as ratios of the propagated standard errors to the overall length (e.g., 1:20,000).

b. Internal accuracy. A loop traverse originating and ending from a single point will have a misclosure when observations (i.e., EDM traverse angles/distances or GPS baseline vectors) are computed forward around the loop back to the starting point. The forward-computed misclosure provides an estimate of the relative or internal accuracy of the observations in the traverse loop, or more correctly, the internal precision of the survey. This is perhaps the simplest method of evaluating the adequacy of a survey. (These point misclosures, usually expressed as ratios, are not the same as relative distance accuracy measures.)

(1) Internal accuracy estimates made relative to a single fixed point are obtained when so-called free, unconstrained, or minimally constrained adjustments are performed. In the case of a single loop, no redundant observations (or alternate loops) back to the fixed point are available. When a series of GPS baseline loops (or network) are observed, then the various paths back to the single fixed point provide multiple position computations, allowing for a statistical analysis of the internal accuracy of not only the position closure but also the relative accuracies of the individual points in the network (including relative distance and azimuth accuracy estimates between these points). The magnitude of these internal relative accuracy estimates (on a free adjustment) determines the adequacy of the control for subsequent design, construction, and mapping work.

(2) Loop traverses are discouraged for most conventional surveys due to potential systematic distance or orientation errors which can be carried through the network undetected. FGCS classification standards for geodetic surveys do not allow traverses to start and terminate at a single point. Such procedures are unacceptable for incorporation into the NGRS network; however, due to many factors (primarily economic), loop traverses or open-ended spur lines are commonly employed in densifying project control for engineering and construction projects. Since such control is not intended for inclusion in the NGRS and usually covers limited project ranges, such practices have been acceptable. Such practices will also be acceptable for GPS surveys performed in support of similar engineering and construction activities.

c. External accuracy. The coordinates (and reference orientation) of the single fixed starting point will also have some degree of accuracy relative to the network in which it is located, such as the NGRS if it was established relative to that system/datum. This "external" accuracy (or inaccuracy) is carried forward in the traverse loop or network; however, any such external variance (if small) is generally not critical to engineering and construction. When a survey is conducted relative to two or more points on an existing reference network, such as USACE project control or the NGRS, misclosures with these fixed control points provide an estimate of the "absolute" accuracy of the survey. This analysis is usually obtained from a final adjustment, usually a fully constrained least squares minimization technique or by other recognized traverse adjustment methods (Transit, Compass, Crandall, etc.).

(1) This absolute accuracy estimate assumes that the fixed (existing) control is superior to the survey being performed, and that any position misclosures at connecting points are due to internal observational errors and not the existing control. This has always been a long-established and practical assumption and has considerable legal basis in property/boundary surveying. New work is rigidly adjusted to existing control regardless of known or unknown deficiencies in the fixed network.

(2) Since the relative positional accuracies of points on the NGRS are known from the NAD 83 readjustment, and GPS baseline vector accuracy estimates are obtained from the individual reductions, variations in misclosures in GPS surveys are not always due totally to errors in the GPS work. Forcing a GPS traverse/network to rigidly fit the existing (fixed) network usually results in a degradation of the internal accuracy of the GPS survey, as compared with a free (unconstrained) adjustment.

11-5. Internal versus External Accuracy

Classical geodetic surveying is largely concerned with absolute accuracy, or the best-fitting of intermediate surveys between points on a national network, such as the NGRS. Alternatively, in engineering and construction surveying, and to a major extent in boundary surveying, relative, or local, accuracies are more critical to the project at hand. Thus, the absolute NAD 27 or NAD 83 coordinates (in latitude and longitude) relative to the NGRS datum reference are of less importance; however, accurate relative coordinates over a given project reach (channel, construction site, levee section, etc.) are critical to design and construction.

a. For example, in establishing basic mapping and construction layout control for a military installation, developing a dense and accurate internal (or relative) control network is far more important than the values of these coordinates relative to the NGRS.

b. On flood control and river and harbor navigation projects, defining channel points must be accurately referenced to nearby shore-based control points. These points, in turn, directly reference boundary/right-of-way points and are also used for dredge/construction control. Absolute coordinates (NGRS/NAD) of these construction and/or boundary reference points are of less importance.

c. Surveys performed with GPS, and final adjustments thereof, should be configured/designed to establish accurate relative (local) project control; of secondary importance is connection with NGRS networks.

d. Although reference connections with the NGRS are desirable and recommended, and should be made where feasible and practicable, it is critical that such connections (and subsequent adjustments thereto) do not distort the internal (relative) accuracy of intermediate points from which design, construction, and/or project boundaries are referenced.

e. Connections and adjustments to distant networks (i.e., NGRS) can result in mixed datums within a project area, especially if not all existing project control has been tied in. This in turn can lead to errors and contract disputes during both design and construction. On existing projects with long-established reference control, connections and adjustments to outside reference datums/networks should be performed with caution. The impacts on legal property and project alignment definitions must also be considered prior to such connections. (See also paragraph 8-3*d*.)

f. On newly authorized projects, or on projects where existing project control has been largely destroyed, reconnection with the NGRS is highly recommended. This will ensure that future work will be supported by a reliable and consistent basic network, while minimizing errors associated with mixed datums.

11-6. Internal and External Adjustments

GPS-performed surveys are usually adjusted and analyzed relative to their internal consistency and external fit with existing control. The internal consistency adjustment (i.e., free or minimally constrained adjustment) is important from a contract compliance standpoint. A contractor's performance should be evaluated relative to this adjustment. The final, or constrained, adjustment fits the GPS survey to the existing network. This is not always easily accomplished since existing networks often have lower relative accuracies than the GPS observations being fit. Evaluation of a survey's adequacy should not be based solely on the results of a constrained adjustment.

11-7. Internal or Geometric Adjustment

This adjustment is made to determine how well the baseline observations fit or internally close within themselves.

(Other EDM distances or angles may also be included in the adjustment.) It is referred to as a free adjustment. This adjustment provides a measure of the internal precision of the survey.

a. In a simplified example, a conventional EDM traverse which is looped back to the starting point will misclose in both azimuth and position, as shown in Figure 11-1. Classical “approximate” adjustment techniques (e.g., Transit, Compass, Bowditch, Crandall) will typically assess the azimuth misclosure, proportionately adjust the azimuth misclosure (usually evenly per station), recompute the traverse with the adjusted azimuths, and obtain a position misclosure. This position misclosure (in X and Y) is then distributed among all the points on the traverse using various weighing methods (distance, latitudes, departures, etc.). Final adjusted azimuths and distances are then computed from grid inverses between the adjusted points. The adequacy/accuracy of such a traverse is evaluated based on the azimuth misclosure and position misclosure after azimuth adjustment (usually expressed as a ratio to the overall length of the traverse).

b. A least squares adjustment of the same conventional loop traverse will end up adjusting the points similarly to the approximate methods traditionally employed. The only difference is that a least squares adjustment simultaneously adjusts both observed angles (or directions) and distance measurements. A least squares adjustment also allows variable weighting to be set for individual angle/distance observations, which is a somewhat more complex process when approximate adjustments are performed. In addition, a least squares adjustment will yield more definitive statistical results of the internal accuracies of each observation and/or point, rather than just the final closure. This includes estimates of the accuracies of individual station X-Y coordinates, relative azimuth accuracies, and relative distance accuracies.

c. A series of GPS baselines forming a loop off a single point can be adjusted and assessed similarly to a conventional EDM traverse loop described in *a* above (see Figure 11-1). The baseline vector components may be computed (accumulated) around the loop with a resultant 3D misclosure back at the starting point. These misclosures (in X, Y, and Z) may be adjusted using either approximate or least squares methods. The method by which the misclosure is distributed among the intermediate points in the traverse is a function of the adjustment weighting technique.

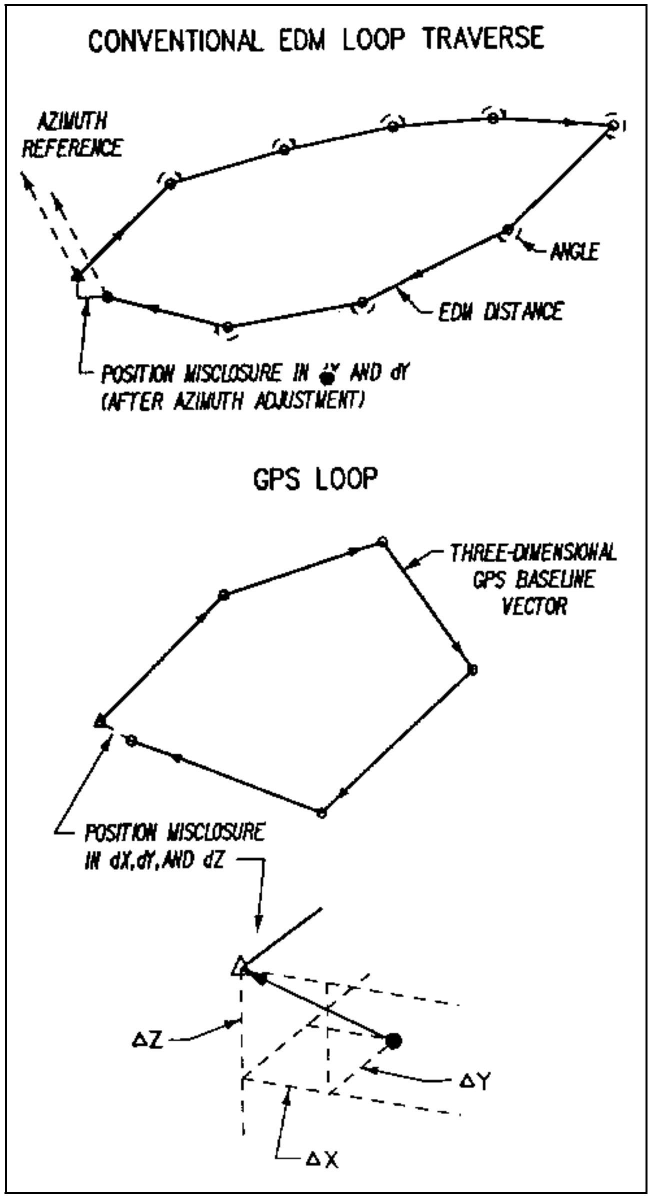


Figure 11-1. Conventional EDM and GPS traverse loops

(1) In the case of a simple EDM traverse adjustment, the observed distances (or position corrections) are weighted as a function of the segment length and the overall traverse length (Compass Rule), or to the overall sum of the latitudes/departures (Transit Rule). Two-dimensional EDM distance observations are not dependent on their direction; that is, a distance’s X- and Y-components are uncorrelated.

(2) GPS baseline vector components (in X, Y, and Z) are correlated due to the geometry of the satellite solution; that is, the direction of the baseline vector is significant. Since the satellite geometry is continuously changing, remeasured baselines will have different correlations between the vector components. Such data are passed down from the baseline reduction software for use in the adjustment.

d. The magnitude of the misclosure (i.e., loop closure) of the GPS baseline vectors at the initial point provides an estimate of the internal precision or geometric consistency of the loop (survey). When this misclosure is divided by the overall length of the baselines, an internal relative accuracy estimate results. This misclosure ratio should not be less than the relative distance accuracy classification intended for the survey, per Table 8-1.

(1) For example, if the position misclosure of a GPS loop is 0.08 m and the length of the loop is 8,000 m, then the loop closure is 0.08/8,000 or 1 part in 100,000 (1:100,000).

(2) When an adjustment is performed, the individual corrections/adjustments made to each baseline (so-called residual errors) provide an accuracy assessment for each baseline segment. A least squares adjustment can additionally provide relative distance accuracy estimates for each line, based on standard error propagation between adjusted points. This relative distance accuracy estimate is most critical to USACE engineering and construction work and represents the primary basis for assessing the acceptability of a survey.

11-8. External or Fully Constrained Adjustment

The internal "free" geometric adjustment provides adjusted positions relative to a single, often arbitrary, fixed point. Most surveys (conventional or GPS) are connected between existing stations on some predefined reference network or datum. These fixed stations may be existing project control points (on NAD 27--SPCS 27) or stations on the NGRS (NAD 83). In OCONUS locales, other local or regional reference systems may be used. A constrained adjustment is the process used to best fit the survey observations to the established reference system.

a. A simple conventional EDM traverse (Figure 11-2) between two fixed stations best illustrates the process by which comparable GPS baseline vectors are adjusted. As with the loop traverse described in paragraph 10-8, the misclosure in azimuth and position between the two fixed end points may be adjusted by any type of approximate or least squares adjustment method. Unlike a loop traverse, however, the azimuth and position misclosures are not wholly dependent on the internal errors in the traverse--the fixed points and their azimuth references are not absolute, but contain relative inaccuracies with respect to one another.

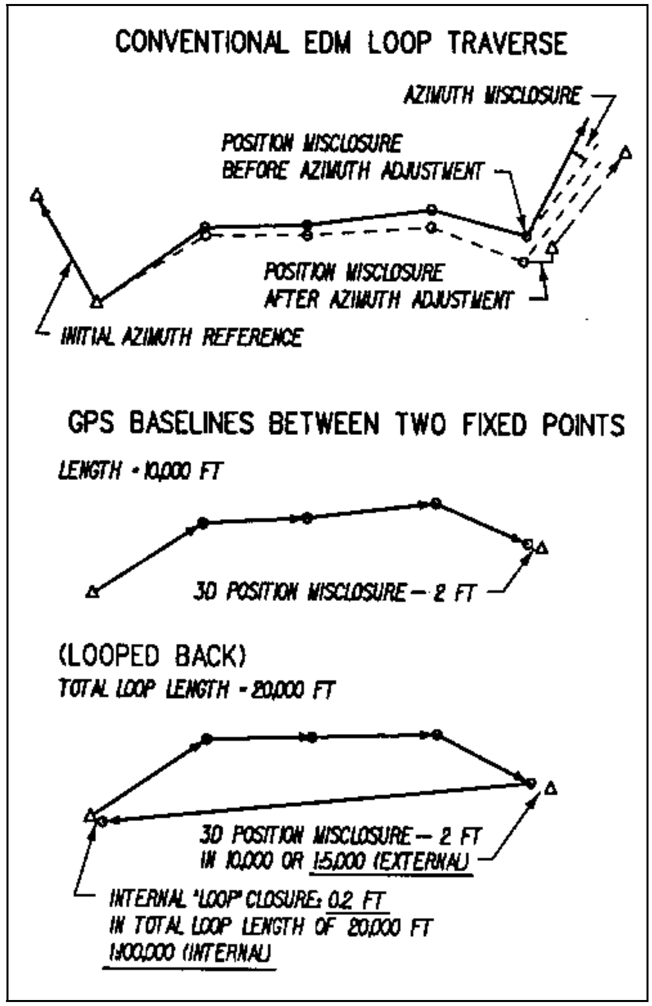


Figure 11-2. Constrained adjustment between two fixed points

b. A GPS survey between the same two fixed points also contains a 3D position misclosure. Due to positional uncertainties in the two fixed network points, this misclosure may (and usually does) far exceed the internal accuracy of the raw GPS observations. As with a conventional EDM traverse, the 3D misclosures may be approximately adjusted by proportionately distributing them over the intermediate points. A least squares adjustment will also accomplish the same thing.

c. If the GPS survey is looped back to the initial point, the free adjustment misclosure at the initial point may be compared with the apparent position misclosure with the other fixed point. In Figure 11-2, the free adjustment loop misclosure is 1:100,000 whereas the misclosure relative to the two network control points is only 1:5,000. Thus, the internal relative accuracy of the GPS survey is on the order of 1 part in 100,000 (based on the misclosure); if the GPS baseline observations are constrained to fit the existing control, the 0.6-m external misclosure must be distributed among the individual baselines to force a fit between the two end points.

(1) After a constrained adjustment, the absolute position misclosure of 0.6 m causes the relative distance accuracies between individual points to degrade. They will be somewhat better than 1:5,000 but far less than 1:100,000. The statistical results from a constrained least squares adjustment will provide estimates of the relative accuracies between individual points on the traverse.

(2) This example also illustrates the advantages of measuring the baseline between fixed network points when performing GPS surveys, especially when weak control is suspected (as in this example).

(3) Also illustrated is the need for making additional ties to the existing network. In this example, one of the two fixed network points may have been poorly controlled when it was originally established, or the two points may have been established from independent networks (i.e., were never connected). A third or even fourth fixed point would be beneficial in resolving such a case.

d. If the intent of the survey shown in Figure 11-2 was to establish 1:20,000 relative accuracy control, connecting between these two points obviously will not provide that accuracy given the amount of adjustment that must be applied to force a fit. For example, if one of the individual baseline vectors was measured at 600 m and the constrained adjustment applied a 0.09-m correction in this sector, the relative accuracy of this segment would be roughly 1:6,666. This distortion would not be acceptable for subsequent design/construction work performed in this area.

e. Most GPS survey networks are more complex than the simple traverse example in Figure 11-2. They may consist of multiple loops and may connect with any number of control points on the existing network. In addition, conventional EDM, angles, and differential leveling measurements may be included with the GPS baselines, resulting in a complex network with many adjustment conditions.

11-9. Partially Constrained Adjustments

In the previous example of the simple GPS traverse, holding the two network points rigidly fixed caused an adverse degradation in the GPS survey, based on the differences between the free (loop) adjustment and the fully constrained adjustment. Another alternative is to perform a semiconstrained (or partially constrained) adjustment of the net. In a partially constrained adjustment, the two network points are not rigidly fixed but only partially fixed in position. The degree to which the existing network points are constrained may be based on their estimated relative accuracies or, if available, their original adjustment positional accuracies (covariance matrices). Partially constrained adjustments are not practicable using approximate adjustment techniques; only least squares will suffice.

a. For example, if the relative distance accuracy between the two fixed network points in Figure 11-2 is approximately 1:10,000, this can be equated to a positional uncertainty between them. Depending on the type and capabilities of the least squares adjustment software, the higher accuracy GPS baseline observations can be best fit between the two end points such that the end points of the GPS network are not rigidly constrained to the original and two control points but will end up falling near them.

b. Adjustment software will allow relative weighting of the fixed points to provide a partially constrained adjustment. Any number of fixed points can be connected to, and these points may be given partial constraints in the adjustment.

c. Performing partially constrained adjustments (as opposed to a fully constrained adjustment) takes advantage of the inherent higher accuracy GPS data relative to the existing network control, which is traditionally weak on many USACE project areas. Less warping of the GPS data (due to poor existing networks) will then occur.

d. A partial constraint also lessens the need for performing numerous trial-and-error constrained adjustments in attempts to locate poor external control points causing high residuals. Fewer ties to the existing network need be made if the purpose of such ties was to find a best fit on a fully constrained adjustment.

e. When connections are made to the NAD 83, relative accuracy estimates of NGRS stations can be obtained from the NGS. Depending on the type of adjustment software used, these partial constraints may be in the form of variance-covariance matrices, error ellipses, or circular accuracy estimates.

11-10. Approximate Adjustments of GPS Networks

Simply constructed GPS networks used for establishing lower order (i.e., Second-Order and lower) USACE control can be effectively adjusted using approximate adjustment techniques, or adjustments which approximate the more rigorous least squares solution. Although least squares solutions may be theoretically superior to approximate methods, the resultant differences between the adjustments are generally not significant from a practical engineering standpoint.

a. Given the high cost of commercial geodetic adjustment software, coupled with the adjustment complexity of these packages, approximate adjustment methods are allowed for in-house and contracted surveys.

b. In practice, any complex GPS survey network may be adjusted by approximate methods. If the main loop/line closures are good, redundant ties to other fixed network points may be used as checks rather than being rigidly adjusted.

c. In some cases it is not cost-effective to perform detailed and time-consuming least squares adjustments on GPS project control surveys requiring only 1:5,000 or 1:10,000 engineering/construction/boundary location accuracy. If internal loop closures are averaging over 1:200,000, then selecting any simple series of connecting baselines for an approximate adjustment will yield adequate resultant positional and relative distance accuracies for the given project requirements. If a given loop/baseline series of say five points miscloses by 0.01 ft over 1,000 m (1:100,000), a case can be made for not even making any adjustment if a relative accuracy of only 1:5,000 is required between points.

d. Any recognized approximate adjustment method may be used to distribute baseline vector misclosures. The method used will depend on the magnitude of the misclosure to be adjusted and the desired accuracy of the survey. These include the following:

(1) Simple proportionate distribution of loop/line position misclosures among the new station coordinates.

(2) Compass Rule.

(3) Transit Rule.

(4) Crandall Method.

(5) No adjustment. Use raw observations if misclosures are negligible.

e. Approximate adjustments are performed using the 3D earth-centered X-Y-Z coordinates. The X-Y-Z coordinates for the fixed points are computed using the transform algorithms shown in *f* below or obtained from the baseline reduction software. Coordinates of intermediate stations are determined by using the baseline vector component differences (ΔX, ΔY, ΔZ) which are obtained directly from the baseline reductions. These differences are then accumulated (summed) forward around a loop or traverse connection, resulting in 3D position coordinate misclosures at the loop nodes and/or tie points. These misclosures are then adjusted by any of the methods in *d* above. GPS vector weighting is accomplished within the particular adjustment method used; there is no need to incorporate the standard errors from the baseline reductions into the adjustment. Internal survey adequacy and acceptance are performed based on the relative closure ratios, as in conventional traversing criteria (see FGCC 1984). Final local datum coordinates are then transformed back from the X-Y-Z coordinates.

f. Given a loop of baseline vectors between two fixed points (or one point looped back on itself), the following algorithms may be used to adjust the observed baseline vector components and compute the adjusted station geocentric coordinates.

(1) Given: Observed baseline vector components ΔX_i, ΔY_i, ΔZ_i for each baseline i (total of n baselines in the loop/traverse). The 3D length of each baseline is l_i, and the total length of the loop/traverse is L.

(2) The misclosures (dx, dy, and dz) in all three coordinates are computed from:

$$\begin{aligned} dx &= X_F + \sum_{i=1}^{i=n} \Delta X_i - X_E \\ dy &= Y_F + \sum_{i=1}^{i=n} \Delta Y_i - Y_E \\ dz &= Z_F + \sum_{i=1}^{i=n} \Delta Z_i - Z_E \end{aligned} \tag{11-1}$$

Where X_F, Y_F, and Z_F are the fixed coordinates of the starting point and X_E, Y_E, and Z_E are the coordinates of the end point of the loop/traverse. (These misclosures would also be used to assess the internal accuracy of the work.)

(3) Adjustments (δx_i, δy_i, δz_i) to each baseline vector component may be computed using either the Compass Rule:

$$\begin{aligned} \delta x_i &= -\,dx \left(\frac{l_i}{L}\right) \\ \delta y_i &= -\,dy \left(\frac{l_i}{L}\right) \\ \delta z_i &= -\,dz \left(\frac{l_i}{L}\right) \end{aligned} \qquad (11\text{-}2)$$

or the Transit Rule:

$$\begin{aligned} \delta x_i &= -\,dx \left(\frac{\Delta X_i}{\sum \Delta X_i}\right) \\ \delta y_i &= -\,dy \left(\frac{\Delta Y_i}{\sum \Delta Y_i}\right) \\ \delta z_i &= -\,dz \left(\frac{\Delta Z_i}{\sum \Delta Z_i}\right) \end{aligned} \qquad (11\text{-}3)$$

(4) The adjusted vector components are computed from:

$$\begin{aligned} \Delta X_i^{\,a} &= \Delta X_i + \delta x_i \\ \Delta Y_i^{\,a} &= \Delta Y_i + \delta y_i \\ \Delta Z_i^{\,a} &= \Delta Z_i + \delta z_i \end{aligned} \qquad (11\text{-}4)$$

(5) The final geocentric coordinates are then computed by summing the adjusted vector components from Equation 11-4 above:

$$\begin{aligned} X_i^{\,a} &= X_F + \Sigma \Delta X_i^{\,a} \\ Y_i^{\,a} &= Y_F + \Sigma \Delta Y_i^{\,a} \\ Z_i^{\,a} &= Z_F + \Sigma \Delta Z_i^{\,a} \end{aligned} \qquad (11\text{-}5)$$

g. Example of an approximate GPS survey adjustment:

(1) Fixed control points from the U.S. Army Yuma Proving Ground GPS Survey (May 1990) (see Figure 11-3):

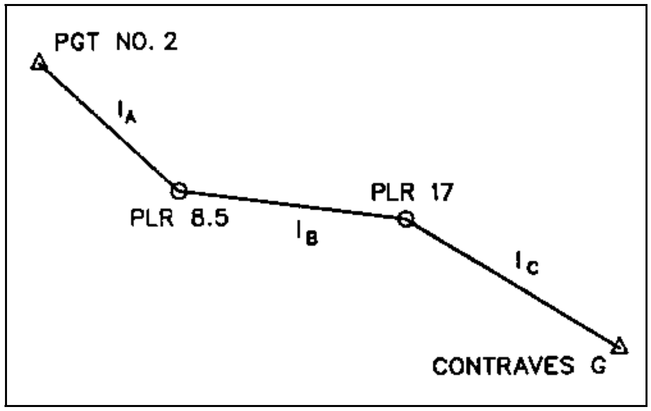


Figure 11-3. Yuma GPS traverse sketch

PGT NO 2:

X_F = (-) 2205 949.0762

Y_F = (-) 4884 126.7921

Z_F = + 3447 135.1550

CONTRAVES G:

X_E = (-) 2188 424.3707

Y_E = (-)4897 740.6844

Z_E = + 3438 952.8159

(XYZ geocentric coordinates were computed from GP-XYZ transform using Equations 11-6 and 11-7 below)

l_a, l_b, l_c = observed GPS baseline vectors (from baseline reductions)

and PLR 8.5 and PLR 17 are the points to be adjusted.

(2) Misclosures in X, Y, and Z (from Equation 11-1):

(-)2205	949.0762 X_F	(-)4884	126.7921 Y_F
+3	777.9104 ΔX_a	(-)6	006.8201 ΔY_a
+7	859.4707 ΔX_b	(-)3	319.1092 ΔY_b
+5	886.8716 ΔX_c	(-)4	288.9638 ΔY_c
-(-)2188	424.3707 X_E	-(-)4897	740.6844 Y_E

$dx = (-)\ 0.4528$ $dy = (-)\ 1.0008$

3447	135.1550 Z_F
(-)6	231.5468 ΔZ_a
+	400.1902 ΔZ_b
(-)2	350.2230 ΔZ_c
- 3438	952.8159 Z_E

$dz = +\ 0.7595$

(3) Linear 3D misclosure:

$$= (0.4528^2 + 1.0008^2 + 0.7595^2)^{1/2} = \underline{1.335 \text{ m}}$$

or 1 part in 25,638.2/1.335 = 1:19,200

(Note: This is a constrained misclosure check, not free)

(4) Compass Rule adjustment:

(a) Compass Rule misclosure distribution:

l_a	= 9,443.869	l_a/L	= 0.368
l_b	= 8,540.955	l_b/L	= 0.333
l_c	= 7,653.366	l_c/L	= 0.299
L	= 25,638.190	Σ	= 1.000

(b) Compass Rule adjustment to GPS vector components using Equation 11-2:

Vector	δ_x	δ_y	δ_z
A	0.1666	0.3683	(-) 0.2795
B	0.1508	0.3333	(-) 0.2529
C	0.1354	0.2992	(-) 0.2271
	(+0.4528)	(+1.0008)	((-)0.7595) Check

(c) Adjusted baseline vectors (Equation 11-4):

Vector	ΔX^a	ΔY^a	ΔZ^a
A	3778.0770	(-)6006.4518	(-)6231.8263
B	7859.6215	(-)3318.7759	399.9373
C	5887.0070	(-)4288.6646	(-)2350.4501

(d) Final adjusted coordinates (Equation 11-5):

	X^a	Y^a
PGT No. 2	(-)2205 949.0762	(-)4884 126.7921
PLR 8.5	(-)2202 170.9992	(-)4890 133.2439
PLR 17	(-)2194 311.3777	(-)4893 452.0198
Contraves G (Check)	(-)2188 424.3707	(-)4897 740.6844

	Z^a	
PGT No. 2	+3447	135.1550
PLR 8.5	+3440	903.3287
PLR 17	+3441	303.2660
Contraves G (Check)	+3438	952.8159

(e) Adjusted geocentric coordinates are transformed to ϕ, λ, h, using Equations 11-9 through 11-13. Geographic coordinates may then be converted to local SPCS (either NAD 83 or NAD 27) project control using USACE program CORPSCON.

(5) Transit Rule adjustment.

(a) Distribution of GPS vector misclosures using Equation 11-3:

$$\Sigma \Delta X_i = 3777.9104 + 7859.4707 + 5886.8716 = 17{,}524.2527$$

Similarly,

$$\Sigma \Delta Y_i = 13{,}614.8931$$

$$\Sigma \Delta Z_i = 8{,}981.9600$$

$$\delta x_i = -dx \left(\frac{\Delta X_i}{\sum \Delta X_i} \right) = -(-) \frac{0.4528}{17{,}524.2527} \ \Delta X_i$$

$$= +2.584 \times 10^5 \quad \Delta X_i$$

Similarly,

$$\delta y_i = +7.351 \times 10^5 \ \Delta Y_i$$
$$\delta z_i = (-)8.456 \times 10^5 \ \Delta Z_i$$

(b) Adjustments to baseline vector components using Transit Rule (Equation 11-3):

Vector	δx	δy	δz
A	0.0976	0.4415	(-)0.5269
B	0.2031	0.2440	(-)0.0338
C	0.1521	0.3153	(-)0.1987
(check)	(0.4528)	(1.0008)	(- 0.7595)

(c) Adjusted baseline vectors (Equation 11-4):

Vector	ΔX^a	ΔY^a	ΔZ^a
A	3 778.0080	(-)6 006.3786	(-)6 232.0737
B	7 859.6738	(-)3 318.8652	+ 400.1564
C	5 887.0237	(-)4 288.6485	(-)2 350.4217

(d) Final adjusted coordinates (Equation 11-5):

	X^a	Y^a
PGT No. 2	(-)2 205 949.0762	(-)4884 126.7921
PLR 8.5	(-)2 202 171.0682	(-)4890 133.1707
PLR 17	(-)2 194 311.3944	(-)4893 452.0359
Contraves G (Check)	(-)2 188 424.3707	(-)4897 740.6844

	Z^a
PGT No. 2	+3447 135.1550
PLR 8.5	+3440 903.0813
PLR 17	+3441 303.2377
Contraves G (Check)	+3438 952.8160

(6) Proportionate distribution adjustment method.

(a) Vector misclosures are simply distributed proportionately over each of the three GPS baselines in the traverse:

$$\delta x = -(-)\frac{0.4528}{3} = +0.1509$$

$$\delta y = -(-)\frac{1.0008}{3} = +0.3336$$

$$\delta z = -(-)\frac{0.7595}{3} = (-)\ 0.2532$$

Vector	ΔX^a	ΔY^a	ΔZ^a
A	3778.0613	(-) 6006.4865	(-) 6231.8000
B	7859.6216	(-) 3318.7756	+ 399.9370
C	5887.0225	(-) 4288.6302	(-) 2350.4762

(b) Final adjusted coordinates:

	X^a	Y^a
PLR 8.5	(-)2202 171.0149	(-)4890 133.2786
PLR 17	(-)2194 311.3933	(-)4893 452.0542

	Z^a
PLR 8.5	+3440 903.3550
PLR 17	+3441 303.2920

Note: Relatively large horizontal (2D) misclosure (1:23,340) may be due to existing control inadequacies, not poor GPS baseline observations.

(c) Variance between adjusted coordinates yields relative accuracies well in excess of 1:20,000; thus, if project control requirements are only 1:10,000, then any of the three adjustment methods may be used.

The recommended method is the Compass Rule.

Fixed coordinates of PGT No. 2 and CONTRAVES G can be on any reference ellipsoid -- NAD 27 or NAD 83.

11-11. Geocentric Coordinate Conversions

The following algorithms for transforming between geocentric and geographic coordinates can be performed in the field on a Hewlett-Packard-style hand-held calculator.

a. Geodetic to Cartesian coordinate conversion. Given geodetic coordinates on NAD 83 (in ϕ, λ, H) or NAD 27, the geocentric Cartesian coordinates (X, Y, and Z) on the WGS 84, GRS 80, or Clarke 1866 ellipsoid are converted directly by the following formulas.

$$X = (R_N + h)\cos\phi\cos\lambda$$
$$Y = (R_N + h)\cos\phi\sin\lambda \qquad (11\text{-}6)$$
$$Z = \left(\frac{b^2}{a^2} R_N + h\right)\sin\phi$$

where

ϕ = latitude

$\lambda = 360^{o} - \lambda_W$ (for CONUS west longitudes)

h = the ellipsoidal elevation. If only the orthometric elevation H is known, then that value may be used.

The normal radius of curvature R_N can be computed from either of the following equations:

$$R_N = \frac{a^2}{\sqrt{a^2\cos^2\phi + b^2\sin^2\phi}} \qquad (11\text{-}7)$$

$$R_N = \frac{a}{\sqrt{1 - e^2\sin^2\phi}} \qquad (11\text{-}8)$$

and

a (GRS 80) = 6,378,137.0 m (semimajor axis)
a (WGS 84) = 6,378,137.0 m
a (NAD 27) = 6,378,206.4 m

b (GRS 80) = 6,356,752.314 1403 m (semiminor axis)
b (WGS 84) = 6,356,752.314 m
b (NAD 27) = 6,356,583.8 m

f (GRS 80) = 1/298.257 222 100 88 (flattening)
f (WGS 84) = 1/298.257 223 563
f (NAD 27) = 1/294.978 698

e^2 (GRS 80) = 0.006 694 380 222 90 (eccentricity squared)
e^2 (WGS 84) = 0.006 694 379 9910
e^2 (NAD 27) = 0.006 768 658

NAD 27 = Clarke Spheroid of 1866
GRS 80 = NAD 83 reference ellipsoid

also

$b = a(1 - f)$

$e^2 = f(2 - f) = (a^2 - b^2) / a^2$

$e^2 = (a^2 - b^2) / b^2$

b. Cartesian to geodetic coordinate conversion. In the reverse case, given GRS 80 X, Y, Z coordinates, the conversion to NAD 83 geodetic coordinates (ϕ, λ, H) is performed using the following noniterative method (Soler and Hothem 1988):

$$\lambda = \arc\tan\frac{Y}{X} \qquad (11\text{-}9)$$

The latitude ϕ and height h are computed using the following sequence. The initial reduced latitude β_o is first computed:

$$\tan\beta_0 = \frac{Z}{p}\left[(1 - f) + \frac{e^2 a}{r}\right] \qquad (11\text{-}10)$$

where

$$p = \sqrt{X^2 + Y^2}$$
$$e^2 = 2f - f^2$$
$$r = \sqrt{p^2 + Z^2}$$

Directly solving for ϕ and h:

$$\tan\phi = \frac{Z(1 - f) + e^2 a\sin^3\beta_0}{(1 - f)(p - ae^2\cos^3\beta_0)} \qquad (11\text{-}11)$$

$$h^2 = (p - a\cos\beta)^2 + (Z - b\sin\beta)^2 \qquad (11\text{-}12)$$

where the final reduced latitude β is computed from

$$\tan\beta = (1 - f)\tan\phi \qquad (11\text{-}13)$$

c. Transforms between other OCONUS datums may be performed by changing the ellipsoidal parameters *a*, *b*, and *f* to that datum's reference ellipsoid.

d. Example geocentric-geographic coordinate transform.

Geographic to geocentric (ϕ, λ, h to X, Y, Z) transform:

(1) Given any point:

ϕ_N = 35° 27' 15.217"

λ_W = 94° 49' 38.107"

λ = 360° - λ_W = 265.1727481°

h = 100 m (N = 0 assumed)

(2) Given constants (WGS 84):

a = 6,378,137 m b = a(1 - f) = 6,356,752.314

f = 1/298.257223563 e^2 = f(2 - f) = 6.694380 x 11^{-3}

$$R_N = a/(1 - e^2 \sin^2 \phi)^{1/2} = \underline{6{,}385{,}332{,}203}$$
$$X = (R_N + h) \cos \phi \cos \lambda = \underline{(-)437{,}710.553}$$
$$Y = (R_N + h) \cos \phi \sin \lambda = \underline{(-)5{,}182{,}990.319}$$
$$Z = \left(\frac{b^2}{a^2} R_N + H \right) \sin \phi = \underline{+3{,}679{,}090.327}$$

e. Geocentric (X, Y, Z) to geographic (ϕ, λ, H) transform.

Inversing the above X, Y, Z geocentric coordinates:

p = $(X^2 + Y^2)^{1/2}$ = 5,201,440.106

r = $(p^2 + Z^2)^{1/2}$ = 6,371,081.918

$$\beta_o = \tan^{-1} \frac{Z}{p} \left[(1 - f) + \frac{e^2 a}{r} \right] = 35.36295229°$$

$$\tan \phi = \frac{Z(1 - f) + e^2 a \sin^3 \beta_o}{(1 - f)(p - ae^2 \cos^3 \beta^o)}$$
$$= 0.712088398$$

ϕ = 35.45422693° = 35° 27' 15.217"

λ = $\tan^{-1}$(Y/X) = 85.17274810° (= 265.17274810°)

λ_W = 360° - λ = 94° 49' 38.107"

β = $\tan^{-1}$ [(1 - f) tan ϕ] = 35.36335663°

h^2 = $(p - a \cos \beta)^2 + (Z - b \sin \beta)^2$
= $(81.458)^2 + (58.004)^2$

h = 99.999 = 100 m

f. North American Datum of 1927 (Clarke Spheroid of 1866). Given a point with SPCS/Project coordinates on NAD 27, the point may be converted to X, Y, Z coordinates for use in subsequent adjustments.

ϕ_N = 35° 27' 15.217"

λ_W = 94° 49' 38.107" h or H = 100 m

(NAD 27 from SPCS X-Y ϕ,λ conversion using USACE program CORPSCON)

a = 6,378,206.4

b = 6,356,583.8

f = 1/294.978698

e^2 = 0.006768658
(NAD 27/Clarke 1866 Spheroid)

$$R_N = \frac{a}{(1 - e^2 \sin^2 \phi)^{1/2}} = \underline{6{,}392{,}765.205}$$

$$X = (R_N + h) \cos \phi \cos \lambda = (-)\ 438{,}220.073 \text{ m}$$
$$Y = (R_N + h) \cos \phi \sin \lambda = (-)\ 5{,}189{,}023.612 \text{ m}$$
$$Z = \left(\frac{b^2}{a^2} R_N + H \right) \sin \phi = +3{,}733{,}466.852 \text{ m}$$

These geocentric coordinates (on NAD 27 reference) may be used to adjust subsequent GPS baseline vectors observed on WGS 84.

11-12. Rigorous Least Squares Adjustments of GPS Surveys

Adjustment of GPS networks on PC-based software is typically a trial-and-error process for both the free and constrained adjustments. When a least squares adjustment is performed on a network of GPS observations, the adjustment software will provide 2D or 3D coordinate accuracy estimates, variance-covariance matrix data for the adjusted coordinates, and related error ellipse data. Most software will provide relative accuracy estimates (length and azimuth) between points. Analyzing these various statistics is not easy, and they are also easily misinterpreted. Arbitrary rejection and readjustment in order to obtain a best fit (or best statistics) must be avoided. The original data reject criteria must be established and justified in a final report document.

a. When a series of loops are formed relative to a fixed point or off another loop, different redundant conditions are formed. (This is comparable to loops formed in conventional differential level nets.) These different loops allow forward baseline vector position computations to be made over different paths. From the different routes (loops) formed, different positional closures at a single fixed point result. These variances in position misclosures from the different routes provide additional data for assessing the internal consistency of the network, in addition to checking for blunders in the individual baselines. The number of different paths, or conditions, is partially related to the number of degrees of freedom in the network.

(1) Multiple observed baseline observations also provide additional redundancy or strength to a line or network since they are observed at two distinct times of varying satellite geometry and conditions. The amount of redundancy required is a function of the accuracy requirements of a particular survey.

(2) Performing a free adjustment on a complex network containing many redundancies is best performed using least squares methods. An example of such a network is shown in Figure 11-4. Approximate adjustment methods are difficult to evaluate when complex interweaving networks are involved.

(3) Baseline reduction vector component error statistics are usually carried down into the least squares adjustment; however, their use is not mandatory for lower order engineering surveys. GPS network least squares adjustments can be performed without all the covariance and correlation statistics from the baseline reduction.

(4) In practice, any station on the network can be held fixed for the free adjustment. The selected point is held fixed in all three coordinates, along with the orientation of the three axes and a network scale parameter. Usually one of the higher order points on the existing network is used.

b. Least squares adjustment software will output various statistics from the free adjustment to assist in detecting blunders and residual outliers in the free adjustment. Most commercial packages will display the normalized residual for each observation (GPS, EDM, angle, elevation, etc.), which is useful in detecting and rejecting residual outliers. The variance of unit weight is also important in evaluating the overall adequacy of the observed network. Other statistics, such as tau, chi-square, confidence levels, histograms, etc., are usually not significant for lower order USACE engineering projects, and become totally insignificant if one is not well versed in statistics and adjustment theory. Use of these statistics to reject data (or in reporting results of an adjustment) without a full understanding of their derivation and source within the network adjustment is ill-advised; they should be "turned off" if they are not fully understood.

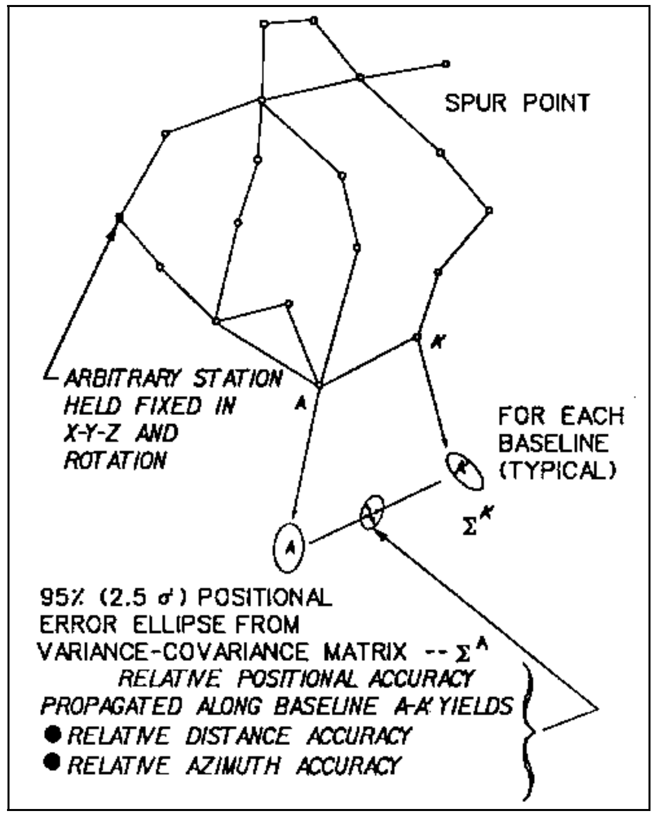


Figure 11-4. Free adjustment of a complex GPS network

c. Relative positional and distance accuracy estimates resulting from a free (unconstrained) geometric adjustment of a GPS network are usually excellent in comparison to conventional surveying methods. Loop misclosures and relative distance accuracies between points will commonly exceed 1:100,000.

d. Relative distance accuracy estimates between points in a network are determined by error propagation of the relative positional standard errors at each end of the line, as shown in Figure 11-4. Relative accuracy estimates may be derived for resultant distances or azimuths between the points. The relative distance accuracy estimates are those typically employed to assess the free (geometric) and constrained accuracy classifications, expressed as a ratio, such as 1:80,000. Since each point in the network will have its particular position variances, the relative distance accuracy propagated between any two points will also vary throughout the network.

(1) The minimum value (i.e., largest ratio) will govern the relative accuracy of the overall project. This minimum value (from a free adjustment) is then compared with the intended relative accuracy classification of the project to evaluate compliance. However, relative distance accuracy estimates should not be rigidly evaluated over short lines (i.e., less than 500 m).

(2) Depending on the size and complexity of the project, large variances in the propagated relative distance accuracies can result.

(3) When a constrained adjustment is performed, the adequacy of the external fixed stations will have a major impact on the resultant propagated distance accuracies, especially when connections are made to weak control systems. Properly weighted partially constrained adjustments will usually improve the propagated distance accuracies.

e. The primary criteria for assessing the adequacy of a particular GPS survey shall be based on the relative distance accuracy results from a minimally constrained free adjustment, not the fully constrained adjustment. This is due to the difficulty in assessing the adequacy of the surrounding network. Should the propagated relative accuracies fall below the specified level, then reobservation would be warranted.

(1) If the relative distance accuracies significantly degrade on a constrained adjustment (due to the inadequacy of the surrounding network), any additional connections to the network would represent a change in contract scope. A large variance of unit weight usually results in such cases.

(2) If only approximate adjustments are performed, then the relative distance accuracies may be estimated as a function of the loop or position misclosures, or the residual corrections to each observed length. For example, if a particular loop or line miscloses by 1 part in 200,000, then individual baseline relative accuracies can be assumed adequate if only a 1:20,000 survey is required.

f. Most commercial and Government adjustment software will output the residual corrections to each observed baseline (or actually baseline vector components). These residuals indicate the amount by which each segment was corrected in the adjustment. A least squares adjustment minimizes the sum of the squares of these baseline residual corrections.

(1) A number of commercial least squares adjustment software packages are available which will adjust GPS networks using standard IBM PC or PC-compatible computers. Those commonly used by USACE Commands include the following:

(a) TURBO-NET™, Geo-Comp, Inc., distributed by Geodetic Enterprises, Inc., PO Box 837, Odessa, FL 33556, (813) 920-4045.

(b) Geo-Lab™, distributed by GEOsurv, Inc., The Baxter Centre, 6-1050 Baxter Road, Ottawa, Ontario, Canada K2C 3P1, (613) 820-4545.

(c) FILLNET™, distributed by Ashtech, Inc., 1156-C Aster Avenue, Sunnyvale, CA, 94086, (408) 249-1314.

(d) ADJUST™, an adjustment program distributed by the National Geodetic Survey Information Center, Rockville, MD 20852.

(e) TRIMNET™, distruibuted by Trimble Navigation, Inc., 645 North Mary Avenue, P.O. Box 3642, Sunnyvale, CA, 94088-3642, (1-800-TRIMBLE).

(f) STAR*NET™, distributed by STARPLUS SOFTWARE, INC., 460 Boulevard Way, Oakland, CA, 94610, (510) 653-4836).

Annotated sample adjustment outputs from two commercial packages are shown in Figures 11-5 and 11-6.

(2) Relative GPS baseline standard errors can be obtained from the baseline reduction output and, in some software (i.e., Geo-Lab), can be directly input into the adjustment. These standard errors, along with their correlations, are given for each vector component (in X, Y, and Z). They are converted to relative weights in the adjustment. FILLNET allows direct input of vector component standard errors in a ±x + y ppm form. Correlations are not used in FILLNET. The following typical input (a priori) weighting is commonly used in FILLNET:

```
ADJUSTMENT STATISTICS SUMMARY
NETWORK = FTM1
TIME = Wed Dec 15 18:13:40 1993


            ADJUSTMENT SUMMARY

  Network Reference Factor = 9.09
  Chi-Square Test (α = 95%) = FAIL
         Degrees of Freedom = 20.00


       GPS OBSERVATIONS
           Reference Factor = 9.09
                          r = 20.00

GPS Solution    1 Reference Factor =   6.08   r =  2.38
GPS Solution    2 Reference Factor =  14.38   r =  2.66
GPS Solution    3 Reference Factor =   9.91   r =  2.06
GPS Solution    4 Reference Factor =   6.65   r =  2.21
GPS Solution    5 Reference Factor =  11.46   r =  2.13
GPS Solution    6 Reference Factor =   3.37   r =  1.96
GPS Solution    7 Reference Factor =   2.52   r =  2.05
GPS Solution    8 Reference Factor =   5.56   r =  2.10
GPS Solution    9 Reference Factor =  11.65   r =  2.45

                   WEIGHTING STRATEGIES:

     GPS OBSERVATIONS:
          No scalar weighting strategy was used

          No summation weighting strategy was used

          Station Error Strategy:
H.I. error = 0.0010
Tribrach error = 0.0010
```

Figure 11-5. TRIMNET sample adjustment output (Sheet 1 of 6)

COORDINATE ADJUSTMENT SUMMARY
NETWORK = FTM1
TIME = Wed Dec 15 18:13:41 1993

Datum = NAD-83
Coordinate System = Geographic
Zone = Global

Network Adjustment Constraints:
3 fixed coordinates in y
3 fixed coordinates in x
3 fixed coordinates in H
3 fixed coordinates in h

POINT NAME		OLD COORDS	ADJUST	NEW COORDS	1.96σ
1 C2PR					
	LAT=	40° 25′ 35.433030″	+0.000000″	40° 25′ 35.433030″	FIXED
	LON=	74° 20′ 41.879000″	+0.000000″	74° 20′ 41.879000″	FIXED
	ELL HT=	-29.8100m	+0.0000m	-29.8100m	FIXED
	ORTHO HT=	3.0400m	+0.0000m	3.0400m	FIXED
	GEOID HT=	-32.8500m	+0.0000m	-32.8500m	FIXED
2 FTM1					
	LAT=	40° 18′ 46.192066″	+0.000003″	40° 18′ 46.192069″	0.011390m
	LON=	74° 02′ 14.692854″	+0.000000″	74° 02′ 14.692854″	0.011542m
	ELL HT=	-22.3923m	-0.5752m	-22.9675m	0.016921m
	ORTHO HT=	0.0000m	+0.0000m	0.0000m	NOT KNOWN
3 MANT					
	LAT=	40° 02′ 18.425950″	+0.000000″	40° 02′ 18.425950″	FIXED
	LON=	74° 03′ 11.673310″	+0.000000″	74° 03′ 11.673310″	FIXED
	ELL HT=	-32.1600m	+0.0000m	-32.1600m	FIXED
	ORTHO HT=	1.1500m	+0.0000m	1.1500m	FIXED
	GEOID HT=	-33.3100m	+0.0000m	-33.3100m	FIXED
4 SIM3					
	LAT=	40° 28′ 06.064930″	+0.000000″	40° 28′ 06.064930″	FIXED
	LON=	74° 00′ 28.941590″	+0.000000″	74° 00′ 28.941590″	FIXED
	ELL HT=	-30.5800m	+0.0000m	-30.5800m	FIXED
	ORTHO HT=	2.1000m	+0.0000m	2.1000m	FIXED
	GEOID HT=	-32.6800m	+0.0000m	-32.6800m	FIXED

Ftm2
LAT 40° 18′ 46.05528″
LON 74° 02′ 14.77218″
EH -22.985 m

Figure 11-5. (Sheet 2 of 6)

OBSERVATION ADJUSTMENT SUMMARY
NETWORK = FTM1
TIME = Wed Dec 15 18:13:43 1993

OBSERVATION ADJUSTMENT (Tau = 2.85)

GPS Parameter Group 1 GPS Observations

Azimuth rotation = -0.2339 seconds 1.96σ = 0.0587 seconds
Deflection in latitude = +0.0470 seconds 1.96σ = 0.1005 seconds
Deflection in longitude = +0.5992 seconds 1.96σ = 0.1960 seconds
Network scale = 0.999995521210 1.96σ = 0.000000288587

OBS#	BLK#/	TYPE	BACKSIGHT/	UDVC/	OBSERVED/	1.96σ/	TAU
	REF#		INSTRUMENT/	UDPG/	ADJUSTED/	1.96σ/	
			FORESIGHT	SBNT	RESIDUAL	1.96σ	
1	1	gpsaz	-**-	-**-	8°12'30.7755"	0.3399"	0.04
	1		FTM1	-**-	8°12'30.7931"	0.1507"	
			SIM3	1	+0.017599"	0.3046"	
2	1	gpsht	-**-	-**-	-7.6308m	0.0495m	0.23
	1		FTM1	-**-	-7.6160m	0.0219m	
			SIM3	1	+0.014794m	0.0444m	
3	1	gpsds	-**-	-**-	17448.3884m	0.0279m	0.34
	1		FTM1	-**-	17448.4006m	0.0126m	
			SIM3	1	+0.012253m	0.0249m	
4	2	gpsaz	-**-	-**-	260°52'34.5985"	0.2105"	0.02
	1		SIM3	-**-	260°52'34.5939"	0.0587"	
			C2PR	1	-0.004628"	0.2022"	
5	2	gpsht	-**-	-**-	+0.8693m	0.0696m	0.19
	1		SIM3	-**-	+0.8516m	0.0282m	
			C2PR	1	-0.017753m	0.0636m	
6	2	gpsds	-**-	-**-	28957.8274m	0.0312m	0.84
	1		SIM3	-**-	28957.8638m	0.0084m	
			C2PR	1	+0.036478m	0.0300m	
7	3	gpsaz	-**-	-**-	8°12'30.5976"	0.3100"	0.50
	1		FTM1	-**-	8°12'30.7931"	0.1507"	
			SIM3	1	+0.195536"	0.2710"	
8	3	gpsht	-**-	-**-	-7.6089m	0.0316m	0.21
	1		FTM1	-**-	-7.6160m	0.0219m	
			SIM3	1	-0.007130m	0.0228m	
9	3	gpsds	-**-	-**-	17448.4119m	0.0265m	0.33
	1		FTM1	-**-	17448.4006m	0.0126m	
			SIM3	1	-0.011284m	0.0233m	
10	4	gpsaz	-**-	-**-	182°32'19.4314"	0.1863"	0.29
	1		FTM1	-**-	182°32'19.3636"	0.0963"	
			MANT	1	-0.067829"	0.1595"	

Figure 11-5. (Sheet 3 of 6)

11	4 gpsht	-**-	-**-	-9.1984m	0.0388m	0.05
	1	FTM1	-**-	-9.1960m	0.0198m	
		MANT	1	+0.002485m	0.0333m	
12	4 gpsds	-**-	-**-	30496.2387m	0.0281m	0.33
	1	FTM1	-**-	30496.2272m	0.0143m	
		MANT	1	-0.011512m	0.0242m	
13	5 gpsaz	-**-	-**-	184°37'13.9839"	0.1196"	0.17
	1	SIM3	-**-	184°37'14.0097"	0.0586"	
		MANT	1	+0.025802"	0.1043"	
14	5 gpsht	-**-	-**-	-1.5901m	0.0399m	0.21
	1	SIM3	-**-	-1.5806m	0.0250m	
		MANT	1	+0.009521m	0.0311m	
15	5 gpsds	-**-	-**-	47890.3245m	0.0284m	0.68
	1	SIM3	-**-	47890.3000m	0.0138m	
		MANT	1	-0.024483m	0.0248m	
16	6 gpsaz	-**-	-**-	330°08'41.8933"	0.1115"	0.05
	1	MANT	-**-	330°08'41.8865"	0.0587"	
		C2PR	1	-0.006765"	0.0947"	
17	6 gpsht	-**-	-**-	+2.4222m	0.0348m	0.23
	1	MANT	-**-	+2.4307m	0.0242m	
		C2PR	1	+0.008509m	0.0250m	
18	6 gpsds	-**-	-**-	49729.7388m	0.0273m	0.07
	1	MANT	-**-	49729.7364m	0.0144m	
		C2PR	1	-0.002441m	0.0232m	
19	7 gpsaz	-**-	-**-	2°31'42.5754"	0.1792"	0.14
	1	MANT	-**-	2°31'42.6064"	0.0960"	
		FTM1	1	+0.031010"	0.1513"	
20	7 gpsht	-**-	-**-	+9.1917m	0.0324m	0.09
	1	MANT	-**-	+9.1951m	0.0198m	
		FTM1	1	+0.003409m	0.0256m	
21	7 gpsds	-**-	-**-	30496.2264m	0.0268m	0.02
	1	MANT	-**-	30496.2272m	0.0143m	
		FTM1	1	+0.000753m	0.0226m	
22	8 gpsaz	-**-	-**-	295°53'30.6165"	0.1904"	0.21
	1	FTM1	-**-	295°53'30.5663"	0.0923"	
		C2PR	1	-0.050255"	0.1665"	
23	8 gpsht	-**-	-**-	-6.7766m	0.0341m	0.33
	1	FTM1	-**-	-6.7642m	0.0224m	
		C2PR	1	+0.012405m	0.0257m	
24	8 gpsds	-**-	-**-	29010.8695m	0.0272m	0.05
	1	FTM1	-**-	29010.8678m	0.0129m	
		C2PR	1	-0.001702m	0.0239m	
25	9 gpsaz	-**-	-**-	295°53'30.6764"	0.2227"	0.37
	1	FTM1	-**-	295°53'30.5663"	0.0923"	
		C2PR	1	-0.110138"	0.2026"	

Figure 11-5. (Sheet 4 of 6)

26	9 gpsht	-**-	-**-	-6.7600m	0.0564m	0.06
	1	FTM1	-**-	-6.7642m	0.0224m	
		C2PR	1	-0.004171m	0.0518m	
27	9 gpsds	-**-	-**-	29010.8399m	0.0319m	0.66
	1	FTM1	-**-	29010.8678m	0.0129m	
		C2PR	1	+0.027863m	0.0291m	

Figure 11-5. (Sheet 5 of 6)

SUMMARY OF COVARIANCES
NETWORK = FTM1
TIME = Wed Dec 15 18:13:45 1993

FROM/ TO	AZIMUTH/ DELTA H	1.96σ 1.96σ	DISTANCE/ DELTA h	1.96σ 1.96σ	HOR PREC
C2PR FTM1	115°41'34" +6.8425m	0.08" 0.0169m	29010.998m -**-	0.0114m -**-	1: 2536313
C2PR MANT	-**- -**-	-**- -**-	-**- -**-	-**- -**-	-**-
C2PR SIM3	-**- -**-	-**- -**-	-**- -**-	-**- -**-	-**-
FTM1 MANT	182°32'20" -9.1925m	0.08" 0.0169m	30496.364m -**-	0.0114m -**-	1: 2675331
FTM1 SIM3	8°12'31" -7.6125m	0.14" 0.0169m	17448.479m -**-	0.0114m -**-	1: 1527827
MANT SIM3	-**- -**-	-**- -**-	-**- -**-	-**- -**-	-**-

Figure 11-5. (Sheet 6 of 6)

```
PROGRAM FILLNET, Version 3.0.00
LICENSED TO: ASHTECH INC.


Fillnet Input File  acts                      40.3    74.1

a =  6378137.000    1/f =  298.2572221   W      Longitude positive WEST

PRELIMINARY COORDINATES:
                      LAT.               LON.         ELEV.    G.H.  CONSTR.

  1          FTM2 40 18 46.25804    74  2 14.14056    47.648   0.000
  2   FFF    MANT 40  2 18.42595    74  3 11.67331   -32.160   0.000
  3   FFF    C2PR 40 25 35.43303    74 20 41.87900   -29.810   0.000
  4   FFF    SIM3 40 28  6.06493    74  0 28.94159   -30.580   0.000
  5          FTM1 40 18 46.19156    74  2 14.69409   -24.152   0.000

GROUP 1, NO. OF VECTORS AND BIAS CONSTRAINTS:

 11      0.000  0.000    0.000  0.000    0.000  0.000    0.000  0.000

VECTORS:
                  DX          DY          DZ       LENGTH    ERROR CODES

FTM1   FTM2      -1.061      -3.124      -3.223      4.612  3 51.0   51.0  2
FTM1   FTM2      -1.059      -3.126      -3.223      4.613  3 51.0   51.0  2
C2PR   MANT   31466.066 -20017.367 -32897.190   49729.603  3 51.0   51.0  3
C2PR   FTM1   27361.521    -755.791   -9612.798  29010.859  3 51.0   51.0  3
MANT   SIM3   -4788.603  30711.771  36432.475   47890.175  3 51.0   51.0  3
MANT   FTM1   -4104.542  19261.593  23284.378   30496.204  3 51.0   51.0  3
SIM3   FTM1     684.063 -11450.182 -13148.091   17448.407  3 51.0   51.0  3
MANT   FTM1   -4104.553  19261.590  23284.377   30496.203  3 51.0   51.0  3
C2PR   SIM3   26677.446  10694.407   3535.282   28957.809  3 51.0   51.0  3
C2PR   FTM1   27361.503    -755.790   -9612.797  29010.842  3 51.0   51.0  3
SIM3   FTM1     684.055 -11450.181 -13148.088   17448.404  3 51.0   51.0  3

 SHIFTS:
   1  -6.254 -14.914 -70.647
   2   0.000   0.000   0.000
   3   0.000   0.000   0.000
   4   0.000   0.000   0.000
   5   0.009   0.035   1.170

ADJUSTED VECTORS, GROUP 1:
                                DX,DY,DZ    V       DN,DE,DU    v      v'

FTM1   FTM2     3213B           -1.060  0.001       -4.214 -0.001   -0.3
                                -3.125 -0.001       -1.876  0.001    0.1
                                -3.223 -0.000       -0.014  0.001    0.2

FTM1   FTM2     3213A           -1.060 -0.001       -4.214  0.001    0.3
                                -3.125  0.001       -1.876 -0.001   -0.1
                                -3.223 -0.000       -0.014 -0.001   -0.2

C2PR   MANT     3203C        31466.069 -0.002   -43116.924  0.019    0.3
                            -20017.389  0.005    24778.270 -0.000   -0.0
                            -32897.156  0.021      -20.476  0.009    0.2

C2PR   FTM1     3203C        27361.531  0.000   -12649.806  0.009    0.2
                              -755.810  0.013    26107.654  0.004    0.1
```

Figure 11-6. FILLNET sample adjustment output (Sheet 1 of 3)

```
                          -9612.775  0.001      53.853 -0.009  -0.2

MANT    SIM3     3203B    -4788.587  0.006    47738.483 -0.015  -0.2
                          30711.758 -0.003     3808.387  0.005   0.1
                          36432.476 -0.015       36.869 -0.006  -0.1

MANT    FTM1     3203B    -4104.538 -0.001    30467.118 -0.010  -0.3
                          19261.579 -0.009     1329.384 -0.003  -0.1
                          23284.381 -0.006       74.329  0.003   0.1

SIM3    FTM1     3203B      684.050 -0.009   -17271.365  0.003   0.1
                         -11450.179 -0.002    -2479.003 -0.009  -0.4
                         -13148.095  0.003       37.460  0.001   0.1

MANT    FTM1     3203C    -4104.538  0.010    30467.118 -0.009  -0.2
                          19261.579 -0.006     1329.384  0.008   0.2
                          23284.381 -0.005       74.329  0.003   0.1

C2PR    SIM3     3203A    26677.482  0.021     4621.559  0.002   0.1
                          10694.369 -0.001    28586.657  0.020   0.6
                           3535.320  0.009       16.393  0.011   0.3

C2PR    FTM1     3203A    27361.531  0.018   -12649.806  0.004   0.1
                           -755.810  0.012    26107.654  0.021   0.6
                          -9612.775 -0.000       53.853 -0.005  -0.1

SIM3    FTM1     3203A      684.050 -0.001   -17271.365 -0.001  -0.1
                         -11450.179 -0.003    -2479.003 -0.002  -0.1
                         -13148.095  0.000       37.460  0.002   0.1

S.E. OF UNIT WEIGHT =      0.278

NUMBER OF -
  OBS. EQUATIONS              33
  UNKNOWNS                    10
  DEGREES OF FREEDOM          23
  ITERATIONS                   0


GROUP  1 ROT. ANGLES (sec.) AND SCALE DIFF. (ppm):

HOR. SYSTEM          0.342   -0.057    0.026    0.101
STD. ERRORS          0.044    0.030    0.025    0.120
XYZ SYSTEM          -0.110    0.178    0.277

ADJUSTED POSITIONS:

                  LAT.              LON.           ELEV.    STD. ERRORS (m)

  1  FTM2    40 18 46.05528    74  2 14.77217    -22.999 0.003 0.004 0.004
  2  MANT    40  2 18.42595    74  3 11.67331    -32.160 0.000 0.000 0.000
  3  C2PR    40 25 35.43303    74 20 41.87900    -29.810 0.000 0.000 0.000
  4  SIM3    40 28  6.06493    74  0 28.94159    -30.580 0.000 0.000 0.000
  5  FTM1    40 18 46.19186    74  2 14.69262    -22.982 0.003 0.003 0.004

ACCURACIES (m):
                                    D. LAT.     D. LON.      VERT.

 FTM1          FTM2                  0.001       0.001       0.001
 FTM1          FTM2                  0.001       0.001       0.001
```

Figure 11-6. (Sheet 2 of 3)

C2PR	MANT	0.000	0.000	0.000
C2PR	FTM1	0.003	0.003	0.004
MANT	SIM3	0.000	0.000	0.000
MANT	FTM1	0.003	0.003	0.004
SIM3	FTM1	0.003	0.003	0.004
MANT	FTM1	0.003	0.003	0.004
C2PR	SIM3	0.000	0.000	0.000
C2PR	FTM1	0.003	0.003	0.004
SIM3	FTM1	0.003	0.003	0.004

```
******************************************************************
****                                                          ****
****                 ESTIMATES OF PRECISION                   ****
****                                                          ****
****      Based on the VECTOR ACCURACIES produced by          ****
****                        FILLNET                           ****
****                                                          ****
****    This is a reasonable estimate of the accuracies       ****
****      of the vectors in the network at 1 SIGMA.           ****
****                                                          ****
******************************************************************
```

VECTOR		LENGTH	PPM(h)	RATIO(h)	PPM(v)	RATIO(v)
FTM1	FTM2	4.613	306.6	1: 3262	216.8	1: 4613
FTM1	FTM2	4.613	306.6	1: 3262	216.8	1: 4613
C2PR	MANT	49729.591	0.0	1: 0	0.0	1: 0
C2PR	FTM1	29010.862	0.1	1: 6837914	0.1	1: 7252715
MANT	SIM3	47890.165	0.0	1: 0	0.0	1: 0
MANT	FTM1	30496.198	0.1	1: 7188001	0.1	1: 7624049
SIM3	FTM1	17448.407	0.2	1: 4112620	0.2	1: 4362102
MANT	FTM1	30496.198	0.1	1: 7188001	0.1	1: 7624049
C2PR	SIM3	28957.832	0.0	1: 0	0.0	1: 0
C2PR	FTM1	29010.862	0.1	1: 6837914	0.1	1: 7252715
SIM3	FTM1	17448.407	0.2	1: 4112620	0.2	1: 4362102
SIM3	FTM1	17448.407	0.2	1: 4112620	0.2	1: 4362102

Figure 11-6. (Sheet 3 of 3)

(a) Fixed: ±3 mm (Lat) ± 5 mm (Long) + 1 ppm ±5 mm (Height) + 1 ppm

(b) Float: ± 6 mm (Lat) ± 10 mm (Long) + 2 ppm ± 10 mm (Height) + 2 ppm

The optimum standard errors shown have been found to be reasonable in standard USACE work where extremely long baselines are not involved. Use of these optimum values is recommended for the first adjustment iteration.

(3) The adequacy of the initial network weighting described in (2) above is indicated by the variance of unit weight (or variance factor in Geo-Lab) which equals the square of the standard error of unit weight (FILLNET). The variance of unit weight should range between 0.5 and 1.5 (or the standard error of unit weight should range between 0.7 and 1.2), with an optimum value of 1.0 signifying realistic weighting of the GPS input observations. A large unit variance (say 5.0) indicates the initial GPS standard errors were too optimistic (low). A low unit variance (say 0.1) indicates the results from the adjustment were better than the assumed GPS baseline precisions used. This unit variance test, however, is generally valid only when a statistically significant number of observations are involved. This is a function of the number of degrees of freedom shown on the adjustment. To evaluate the adequacy of the unit weight, a test such as chi-square in Geo-Lab is performed. Failure of such a test indicates the variance factor statistic may not be statistically valid, including any rejections made using this value.

(4) The input standard errors can easily be juggled in order to obtain a variance of unit weight near 1.0. This trial-and-error method is generally not a good practice. If the input weights are changed, they should not be modified beyond reasonable levels (e.g., do not input a GPS standard error of ±50 + 50 ppm in order to get a good unit variance). If input standard errors are modified, these modifications should be the same for all lines, not just selected ones. Any such modifications of a priori standard errors must be justified in the adjustment report.

(5) Changing the magnitude of the input standard errors/weights will not change the adjusted position or residual results in a free adjustment provided all weight changes are made equally. Although the reference variance will change, the resultant precisions (relative line accuracies) will not change. (This is not true in a constrained adjustment.) Therefore, the internal accuracy of a survey can be assessed based on the free adjustment line accuracies regardless of the initial weighting or variance of unit weight.

(6) The magnitude of the residual corrections shown in the sample adjustments may be assessed by looking for blunders or outliers; however, this assessment should be performed in conjunction with the related normalized residual (FILLNET) or standardized residual (Geo-Lab) statistic. This statistic is obtained by multiplying the residual by the square root of the input weight (the inverse of the square of the standard error). If the observations are properly weighted, the normalized residuals should be around 1.0. Most adjustment software will flag normalized residuals which exceed selected statistical outlier tests. Such flagged normalized residuals are candidates for rejection. A rule-of-thumb reject criterion should be set at three times the standard error of unit weight, again provided that the standard error of unit weight is within the acceptable range given in (3) above. All rejected GPS observations must be justified in the adjustment report clearly describing the test used to remove the observation from the file.

(7) Error ellipses, or 3D error ellipsoids, generated from the adjustment variance-covariance matrices for each adjusted point in Geo-Lab are also useful in depicting the relative positional accuracy. The scale of the ellipse may be varied as a function of the 2D deviation. Usually a 2.45-σ, or 95 percent, probability ellipse is selected for output. The size of the error ellipse will give an indication of positional reliability, and the critical relative distance/azimuth accuracy estimate between two adjacent points is a direct function of the size of these positional ellipses.

(8) The relative distance accuracy estimates (i.e., relative station confidence limits in Geo-Lab and estimates of precision in FILLNET) are used to evaluate acceptability of a survey. This is done using a free adjustment. The output is shown as a ratio (FILLNET) or in parts per million (Geo-Lab). Note that FILLNET uses a 1-σ line accuracy. The resultant ratios must be divided by 2 in order to equate them to FGCS 95 percent criteria. Geo-Lab is set to default to the 95 percent level.

(9) Further details on these statistical evaluations are beyond the scope of this manual. Technical references listed under paragraph A-1 should be consulted.

g. The following is a summary of a network adjustment sequence recommended by the NGS for surveys which are connected with the NGRS:

(1) A minimally constrained 3D adjustment is done initially as a tool to validate the data, check for blunders and systematic errors, and to look at the internal consistency of the network.

(2) A 3D horizontal constrained adjustment is performed holding all previously published horizontal control points fixed and one height constraint. If the fit is poor, then a readjustment is considered. All previous observations determining the readjusted stations are considered in the adjustment.

(3) A fully constrained vertical adjustment is made to determine the orthometric heights. All previously published benchmark elevations are held fixed along with one horizontal position in a 3D adjustment. Geoid heights are predicted using the latest model.

(4) A final free adjustment is performed in which relative accuracy estimates are computed.

11-13. Evaluation of Adjustment Results

A survey shall be classified based on its horizontal point closure ratio, as indicated in Table 11-1 or the vertical elevation difference closure standard given in Table 11-2.

Table 11-1
USACE Point Closure Standards for Horizontal Control Surveys

USACE Classification	Point Closure Standard (Ratio)
Second Order Class I	1:50,000
Second Order Class II	1:20,000
Third Order Class I	1:10,000
Third Order Class II	1: 5,000
4th Order - Construction Layout	1: 2,500 - 1:20:000

Table 11-2
USACE Point Closure Standards for Vertical Control Surveys

USACE Classification	Point Closure Standard (mm)
Second Order Class I	$6mm\sqrt{K}$
Second Order Class II	$8mm\sqrt{K}$
Third Order	$12mm\sqrt{K}$
4th Order - Construction Layout	$24mm\sqrt{K}$

($\sqrt{K}$ is square root of distance K in kilometers)

a. Horizontal control standards. The horizontal point closure is determined by dividing the linear distance misclosure of the survey into the overall circuit length of a traverse, loop, or network line/circuit. When independent directions or angles are observed, as on a conventional survey (i.e., traverse, trilateration, or triangulation), these angular misclosures may optionally be distributed before assessing positional misclosure. In cases where GPS vectors are measured in geocentric coordinates, then the 3D positional misclosure is assessed.

(1) Approximate surveying. Approximate surveying work should be classified based on the survey's estimated or observed positional errors. This would include absolute GPS and some differential GPS techniques with positional accuracies ranging from 10 to 150 ft (2DRMS). There is no order classification for such approximate work.

(2) Higher order surveys. Requirements for relative line accuracies exceeding 1:50,000 are rare for most USACE applications. Surveys requiring accuracies of First-Order (1:100,000) or better should be performed using FGCS standards and specifications, and must be adjusted by the NGS.

(3) Construction layout or grade control (Fourth-Order). This classification is intended to cover temporary control used for alignment, grading, and measurement of various types of construction, and some local site plan topographic mapping or photo mapping control work. Accuracy standards will vary with the type of construction. Lower accuracies (1:2,500 - 1:5,000) are acceptable for earthwork, dredging, embankment, beach fill, and levee alignment stakeout and grading, and some site plan, curb and gutter, utility building foundation, sidewalk, and small roadway stakeout. Moderate accuracies (1:5,000) are used in most pipeline, sewer, culvert, catch basin, and manhole stakeout, and for general residential building foundation and footing construction, major highway pavement, and concrete runway stakeout work. Somewhat higher accuracies (1:10,000 - 1:20,000) are used for aligning longer bridge spans, tunnels, and large commercial structures. For extensive bridge or tunnel projects, 1:50,000 or even 1:100,000 relative accuracy alignment work may be required. Vertical grade is usually observed to the nearest 0.005 m for most construction work, although 0.04-m accuracy is sufficient for riprap placement, grading, and small-diameter-pipe placement. Construction control points are typically marked by semi-permanent or temporary monuments (e.g., plastic hubs, P-K nails, wooden grade stakes). Control may be established by short, nonredundant spur shots, using total stations or GPS, or by single traverse runs between two existing permanent control points. Positional accuracy

will be commensurate with, and relative to, that of the existing point(s) from which the new point is established.

b. Vertical control standards. The vertical accuracy of a survey is determined by the elevation misclosure within a level section or level loop. For conventional differential or trigonometric leveling, section or loop misclosures (in millimeters) shall not exceed the limits shown in Table 11-2, where the line or circuit length (K) is measured in kilometers. Fourth-Order accuracies are intended for construction layout grading work. Procedural specifications or restrictions pertaining to vertical control surveying methods or equipment should not be over-restrictive.

11-14. Final Adjustment Reports and Submittals

a. A variety of free and/or constrained adjustment combinations may be specified for a contracted GPS survey. Specific stations to be held fixed may be indicated or a contractor may be instructed to determine the optimum adjustment, including appropriate weighting for constrained points. When fixed stations are to be partially constrained, then appropriate statistical information must be provided--either variance-covariance matrices or relative positional accuracy estimates which may be converted into approximate variance-covariance matrices in the constrained adjustment. All rejected observations will be clearly indicated, along with the criteria/reason used in the rejection.

b. When different combinations of constrained adjustments are performed due to indications of one or more fixed stations causing undue biasing of the data, an analysis shall be made as to a recommended solution which provides the best fit for the network. Any fixed control points which should be readjusted to anomalies from the adjustment(s) should be clearly indicated in a final analysis recommendation.

c. The final adjusted horizontal and/or vertical coordinate values shall be assigned an accuracy classification based on the adjustment statistical results. This classification shall include both the resultant geodetic/Cartesian coordinates and the baseline differential results. The final adjusted coordinates shall state the 95 percent confidence region of each point and the accuracy in parts per million between all points in the network. The datum and/or SPCS will be clearly identified for all coordinate listings.

d. Final report coordinate listings may be required on hard copy as well as on a specified computer media.

e. It is recommended that a scaled plot be submitted with the adjustment report showing the proper locations and designations of all stations established.

Chapter 12
Estimating Costs for Contracted GPS Surveys

12-1. General

Developing cost estimates for GPS surveys is not markedly different from estimating conventional traverse or topographic mapping surveys. Similar production factors directly affect the ultimate cost: number of available GPS receiver units, daily productivity rates, survey accuracy criteria, network redundancy requirements, and required observation time per station. These factors are discussed in detail in previous chapters of this manual. Once the number of GPS observations for a given project has been determined, then the total field survey time and subsequent costs can be computed. Office data reduction and adjustment functions are performed and cost estimated identically to that of conventional survey work. The explanations herein regarding procurement policies and practices describe only the framework within which cost estimates are used. For detailed guidance on procurement policies and practices, refer to the appropriate procurement regulations.

12-2. Hired Labor Surveys

Developing cost estimates for USACE field forces engaged in GPS surveys is performed similarly to that of conventional topographic survey work. Normally, an average daily rate of personnel, travel, per diem, and equipment is established. The GPS instrumentation rental rate is established at the time of purchase and is periodically updated based on actual utilization rates as charged against projects. Fringes, technical indirect, and direct overhead costs are added to a field crew's direct labor. The GPS survey crew rate should be recomputed at least annually, or more often if GPS instrumentation and other plant rental rates change significantly.

12-3. Contracted GPS Survey Services

In accordance with current laws and regulations, GPS surveying services must be procured using qualification-based selection procedures in accordance with PL 92-582 (Brooks Act). GPS services may be included as part of a fixed-price (single project scope) A-E design contract or included as a line item on an indefinite delivery type (IDT) surveying and mapping A-E contract. In some instances, a fixed-scope GPS service contract may be issued. In all cases, GPS surveying services will be negotiated as part of the A-E selection process; therefore, a Government cost estimate for these services must be prepared in advance of formal negotiations with the contractor.

a. *Contract types.* Fixed-scope GPS service contracts are not common; in most cases, USACE Commands obtain GPS services via the IDT contracting methods. One or more delivery orders may be placed against the IDT contract for specific projects. An overall contract threshold is established--currently $750,000 per year/contract; thus, the accumulation of individual orders cannot exceed this limit. Individual orders placed against the basic contract are normally limited to $150,000. The term of an IDT contract is usually set at 1 year; however, an option for year extensions may be authorized. Separate project scopes are written and negotiated for each order. The unit prices established in the basic IDT contract are used as a basis for estimating and negotiating each delivery order. The basic unit prices (U/P) in an IDT contract are established as part of the A-E acquisition and negotiation process; therefore, a Government cost estimate for these services must be prepared in advance of formal negotiations with the contractor. These basic unit prices must adequately represent the anticipated work over the course of the IDT contract--typically a 1-year period. (Separate rates are negotiated for additional option years.) Deficiencies in these unit rates will impact subsequent delivery order negotiations.

b. *Unit price basis.* A number of methods are used for scheduling GPS services in a fixed-price or IDT contract. The daily rate basis is the cost for a GPS field crew (including all instrumentation, transport, travel, and overhead) over a nominal 8-hr day. This rate method is normally used only on IDT contracts. This pricing method has advantages and drawbacks which need to be considered prior to determining.

(1) A daily crew rate estimating basis is the preferred unit price basis in estimating contracted GPS services for both fixed-price and IDT contracts. It provides the most flexibility for IDT contracts, especially when individual project scopes are expected to vary widely. It is, therefore, considered a more accurate method of determining costs for individual delivery orders. One disadvantage is that a detailed independent government estimate (IGE) must be developed for each delivery order placed against an IDT contract.

(2) The daily rate for a GPS surveying crew must be estimated using the following USACE-directed detailed

analysis method. The crew personnel size, number of GPS receivers deployed, vehicles, etc., must be explicitly indicated in the contract specifications, with differences resolved during negotiations. Options to add additional GPS receiver units (along with personnel and/or transport) must be accounted for in the estimate and unit price schedule. The seven-item breakdown for estimating costs is listed in Table 12-1.

Table 12-1
Factors for Estimating Costs

Item	Description
I	Direct labor or salary costs of GPS survey technicians: includes applicable overtime or other differentials necessitated by the observing schedule.
II	Overhead on Direct Labor.
III	G&A Overhead Costs (on Direct Labor).
IV	Material Costs.[1]
V	Travel and Transportation Costs: crew travel, per diem, etc. Includes all associated costs of vehicles used to transport GPS receivers.[1]
VI	Other Costs: includes survey equipment and instrumentation, such as GPS receivers. GPS receiver costs should be amortized down to a daily rate, based on average utilization rates, expected life, etc. Exclude all instrumentation and plant costs covered under G&A, such as interest.[1]
VII	Profit (To be computed/negotiated on individual delivery orders per EFARS Part 15).

1. Government audit must confirm if any of these direct costs are included in overhead.

(3) A typical contract price schedule using the daily rate basis is shown in Table 12-2. This schedule may be modified as necessary to reflect larger GPS receiver and personnel inventories.

(4) Another advantage of a daily rate basis unit of measure (U/M) is that it is not dependent on the type or order of accuracy of the GPS survey being performed. Either static or kinematic GPS surveys can be estimated and negotiated using this cost basis.

12-4. Verification of Contractor Cost or Pricing Data

Regardless of the cost rate method used, it is essential (but not always required) that a cost analysis, price analysis, and field pricing support audit be employed to verify all cost or pricing data submitted by a contractor, in particular, actual GPS instrumentation utilization rates and reduced costs per day. GPS equipment and instrumentation costs represent a major portion of a field crew's costs, and these cost rates are currently extremely variable. Some GPS operation and maintenance costs may be direct, or portions may be indirectly included in a firm's General and Administrative (G&A) overhead account. In some instances, a firm may lease/rent GPS equipment in lieu of purchase. Rental rates average 10 to 15 percent per month of the purchase cost, or $4,000 to $6,000 per month (1994). Rental would be economically justified only on limited scope projects and if the equipment is deployed on a full-time basis. Whether the GPS equipment is rented or purchased, the primary (and most variable) factor is the GPS equipment's actual utilization rate, or number of actual billing days to clients over a year. Only a detailed audit and cost analysis can establish such rates and justify modifications to the usually rough assumptions used in the IGE. In addition, an audit will establish any nonproductive labor/costs which are transferred to a contractor's G&A. Given the highly changing equipment costs and utilization rates in this new technology, failure to perform a detailed cost analysis and field pricing support audit on contracted GPS services will make the IGE difficult to substantiate.

12-5. Sample Cost Estimate for Contracted GPS Survey Services

The following cost computation is representative of the procedure used in preparing the IGE for an A-E contract. It is developed for a two-receiver, two-man, two-vehicle GPS field survey crew and based on a standard 8-hr workday. Larger crew/receiver size estimates would be performed similarly. Costs and overhead percentages are shown for illustration only--they are subject to considerable geographic-, project-, and contractor-dependent variation (e.g., audited G&A rates could range from 50 to 200 percent). GPS instrumentation rates are approximate (1994) costs. Associated costs for GPS receivers, such as insurance, maintenance contracts, interest, etc., are presumed to be indirectly factored into a firm's G&A overhead account. If not, then such costs must be directly added to the basic equipment depreciation rates shown. Other equally acceptable accounting methods for developing daily costs of equipment may be used. Equipment utilization estimates in an IGE must be subsequently revised (during negotiations) based on actual rates as determined from a detailed cost analysis and field price support audits.

Table 12-2
Daily Rate Basis Contract Schedule

Item	Description	Quan	U/M	U/P	Amount
0001	Registered/Licensed Land Surveyor -- Office	[1]	Day		
0002	Registered/Licensed Land Surveyor	[1]	Day		
0003	Civil Engineering Technician -- Field Party Supervisor (Multiple Crews)	[1]	Day		
0004	Engineering Technician (Draftsman) -- Office	[1]	Day		
0005	Supervisory GPS Survey Technician (Field)	[1]	Day		
0006	Surveying Technician -- GPS Instrumentman/Recorder	[1]	Day		
0007	Surveying Aid -- Rodman/Chainman {Conventional Surveys}	[1]	Day		
0008	[Two][Three][Four][___]- Man GPS Survey Party				
	[___] GPS Receiver(s)				
	[___] Vehicle(s)				
	[___] Computer(s)				
0009	Additional GPS Receiver	[1]	Day		
	{Add Item 0006 Observers as Necessary}	[1]	Day		
0010	Station Monuments [Disk Type] [Construction Materials]	[1]	EA		
0011	Professional Geodesist Computer (office)	[1]	Day		
0012					
0013					

a. *Basic daily crew rate cost estimate.*

(1) Direct Labor.

Supervisory Survey Technician (GPS Observer) @ $20,000/year or $77/day
Survey Technician @ $16,000/year or $62/day

Total direct labor: $139/day

(2) Overhead on direct labor:

@ 30% of direct labor $42/day

(3) G&A overhead:

@ 100% of direct labor $139/day

(4) Materials and supplies: $20/day

(5) Travel and transportation expenses:

Vehicle depreciation:
$17K base @ 5 years @ 220 days/year $15/day

Operation and maintenance (fuel, oil, etc.) $15/day

Total: Two vehicles @ $30/day ea $60/day

Per Diem: average assumed for IDT locale; rate not to exceed published General Services Administration (GSA)/Joint Travel Regulation (JTR) levels

Total: Two men @ $50/day each $100/day

(6) Other costs:

(Miscellaneous survey instrumentation/equipment, tools and equipment (T&E), etc., normally included in G&A overhead.)

GPS receivers (2 each) plus 386-based field computer

Receivers: 2 @ $20K ea	$40,000
Computer + software:	$10,000
Total:	$50,000

5-year depreciation base --
assumed average utilization
of 200 days per year with
maintenance included in G&A rate

Total: $50K @ 5 years @ $200/day) $50/day

(7) Profit: Profit is not computed on the basic contract but is determined for each separate order based on the guidance contained in Part 15 of the EFARS.

Total Estimated Rate:	$550/day

b. Additional GPS receiver.

Direct Labor (Survey Technician)	$62/day
Overhead on direct labor @ 30%	$19/day
G&A @ 100%	$62/day
Material and supplies	$5/day
Travel and transportation:	
Vehicle	$30/day
Per diem	$50/day
Other costs: GPS Receiver	
One receiver $20K @ 5 years @ 200 days/year	$20/day
Total:	$248/day

c. Travel and per diem. The contract schedule must equitably account for actual travel and per diem expenses if a constant temporary duty locale is not involved, or if the per diem rate varies considerably from that estimated for an IDT contract. Some USACE Commands include crew per diem as a separate line item on the schedule or develop a schedule containing local and travel crew rates.

d. Delivery orders. Since unit prices (either daily rates or work unit rates) have been established in the basic contract, each such delivery order is negotiated strictly for effort. The negotiated fee on a delivery order is then a straight mathematical procedure of multiplying the agreed-upon effort (time or unit of measure quantity) against the unit prices, plus an allowance for profit. Thus, an IGE is required for each order placed, along with a detailed profit computation, documented records of negotiations, etc. The scope is attached to a DD 1155 order placed against the basic contract. The process for estimating the time to perform any particular survey function, in a given project, is totally dependent upon the knowledge and personal field experience of the Government estimator.